H-3 CHAPEL

a. Introduction.

This design example illustrates the seismic design of a church building. The layout of the building is based on a typical military church structure.

(1) Purpose. The purpose of this example is to illustrate the design of a representative military building in an area of high seismicity, using the provisions of FEMA 302 as modified by this document.

(2) Scope. The scope of this example problem includes; the design of all major structural members such as steel gravity and moment framing, reinforced concrete shear walls and horizontal steel pipe bracing. The design of the foundations, nonstructural elements and their connections, and detail design of some structural elements such as reinforced concrete slabs (roof and slab on grade) were not considered part of the scope of this problem and are therefore not included (See Problem H-2 for the design of concrete floor and roof slab and Problem H-4 for the detailed design of steel moment connections)

b. Building Description.

(1) Function. This building functions as a Chapel with a capacity of more than 300 people.

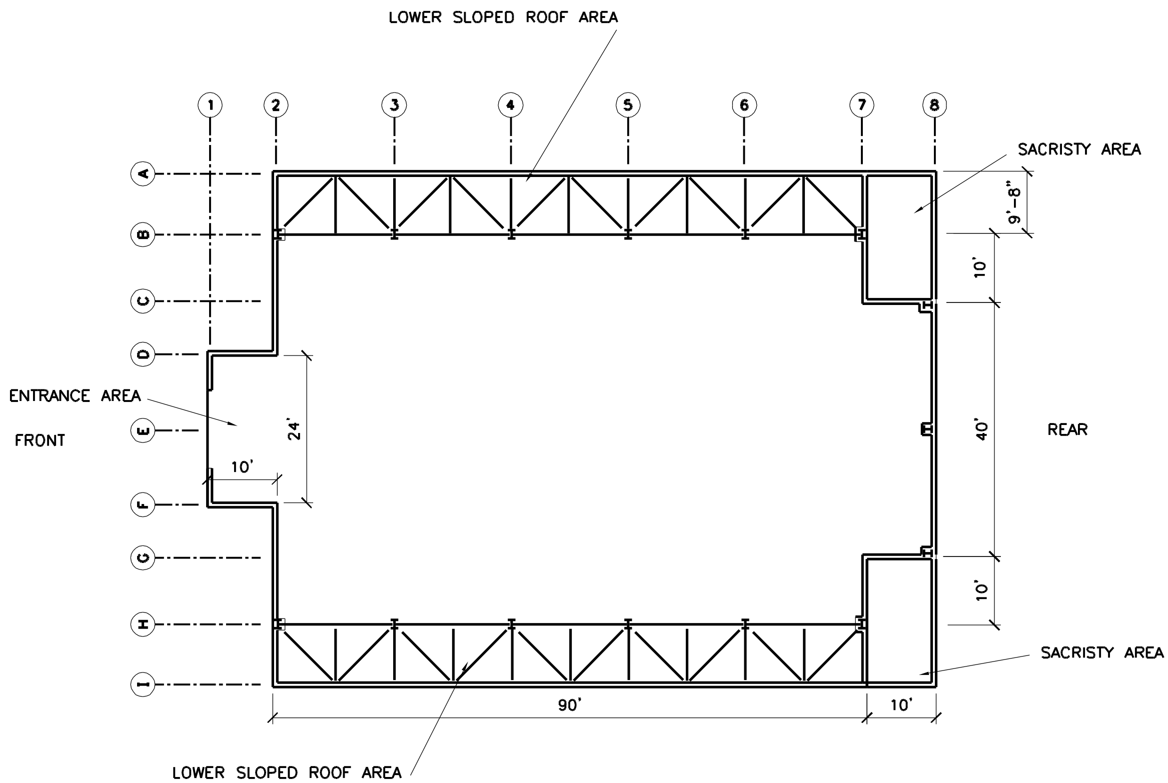
(2) Seismic Use Group. The Seismic Use Group is determined from Table 4-1. The primary occupancy of this structure is public assembly with a capacity greater than 300 persons. This type of occupancy places the building in Seismic Use Group II, Special Occupancy Structures. With the Seismic Use Group known, the Structural System Performance Objectives are obtained from Table 4-4. Structures in Seismic Use Group II are to be designed for Performance Level 2, Safe Egress. Ground Motion A (2/3 MCE) is to be used for Performance Objective 2A. The Minimum Analysis Procedure to be used is the Linear Elastic with R Factors and Linear Elastic with m Factors. The structure is designed first for Performance Objective 1A following the steps laid out in Table 4-5. After completion of the preliminary design, the enhanced performance objectives outlined in Table 4-6 for Performance Objective 2A are checked and the building design updated accordingly to meet those objectives.

(3) Configuration. The main chapel area has a high roof area (roof at ridge is 33'-3" high or 10.14m). There are low roof areas (10' in height or 3.05m) that run 90' (27.45m) on each side of the main open area. There are two sacristy areas at the rear end of the building that measure 10' x 19'-8" by 10' high (3.05m x 6.00m by 3.05m high)

(4) Structural systems. Steel transverse moment frames support the gravity loads from the high roof area. Metal decking spans over purlins spaced at 10' (3.05m), and the purlins span to the steel moment frames (spaced at 18' or 5.49m o.c.). In the low sloped roof areas that run parallel to the main high roof area, gravity loads are supported by metal decking, the decking spans between the shear walls along lines A & I and the window walls along lines B & H. The upper level concrete window/shear walls along lines B & H are supported by beams that span in the longitudinal direction between the columns of the transverse moment frames. Gravity loads at the sacristy areas are supported by reinforced concrete slabs (slab design not included in scope of problem).

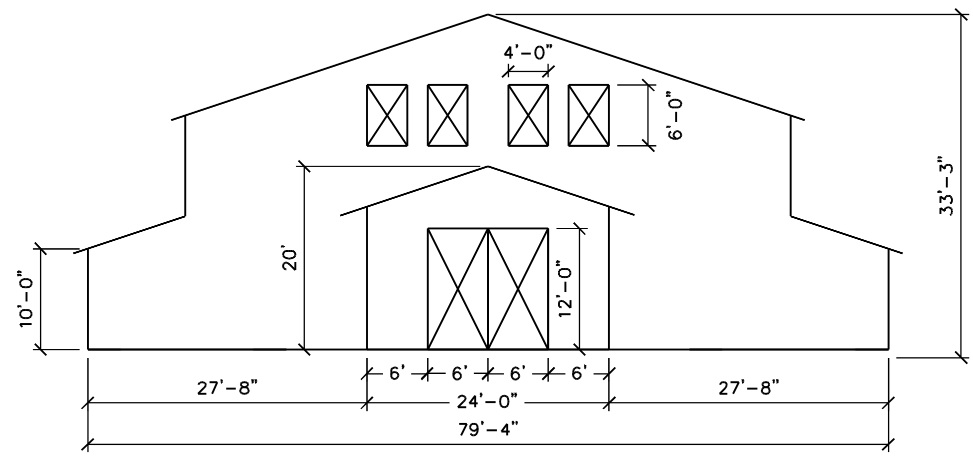
The primary lateral force resisting elements for this structure consists of specially reinforced concrete shear walls. In the longitudinal direction the concrete shear walls resist the entire shear force. Seismic forces from the upper roof area are transferred from the diaphragm to the shear walls along lines B & H. These walls are supported by steel beams at the level of the lower sloped roof diaphragms and are not continuous to the ground. Horizontal pipe bracing transfers the shear from these walls to the exterior shear walls. In the transverse direction lateral forces are also resisted by a combination of concrete shear walls and steel moment frames. The upper roof diaphragm is assumed to act as a flexible diaphragm. Therefore, inertial forces are resisted by the concrete shear walls and steel moment frames based on tributary areas. The moment frames are assumed to be braced by the horizontal bracing at the level of the lower sloped roof areas (along wall lines B & H). The horizontally braced diaphragms of the low sloped area are assumed to act as rigid diaphragms (the diaphragm action falls between flexible and rigid. Analyzing the diaphragms as rigid produce was found to produce the most conservative design for the shear walls and horizontal bracing). The sacristy roof diaphragms are composed of reinforced concrete slabs which are assumed to act as rigid diaphragms.

(5) Choice of materials. The concrete shear walls are chosen due to their high stiffness and strength. The transverse steel moment frames that support the high roof area allows for a large open space with a high ceiling. Large shear forces are required to be transferred in the longitudinal direction from the shear walls along lines B & H to the exterior walls along lines A & I. Horizontal pipe bracing consisting of extra strong sections were chosen to transfer these large forces to the exterior shear walls.



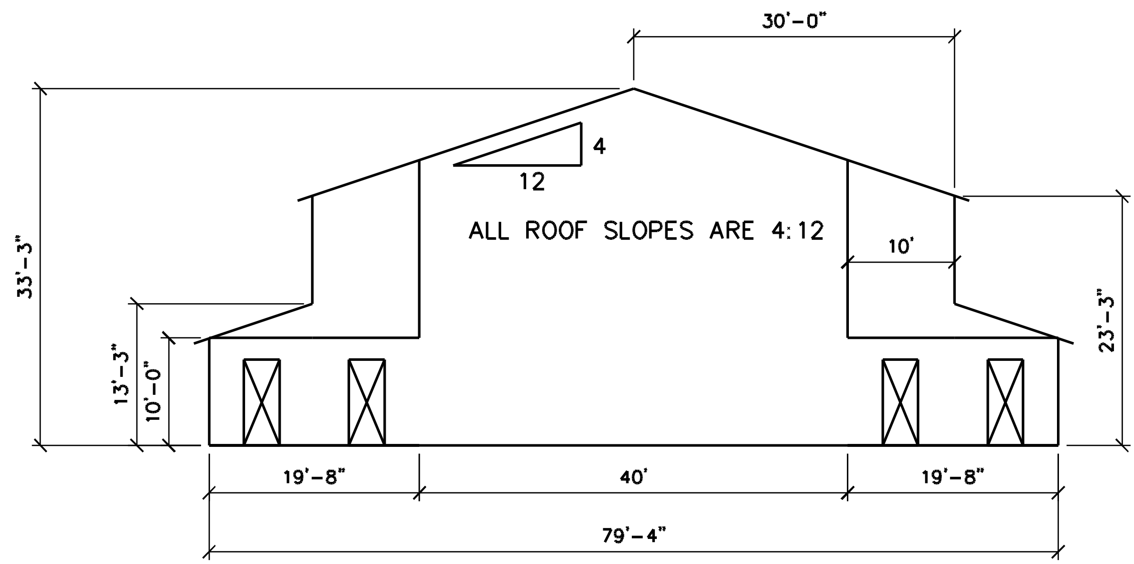
1 foot = 0.305 m
1 inch = 25.4 mm

FLOOR PLAN

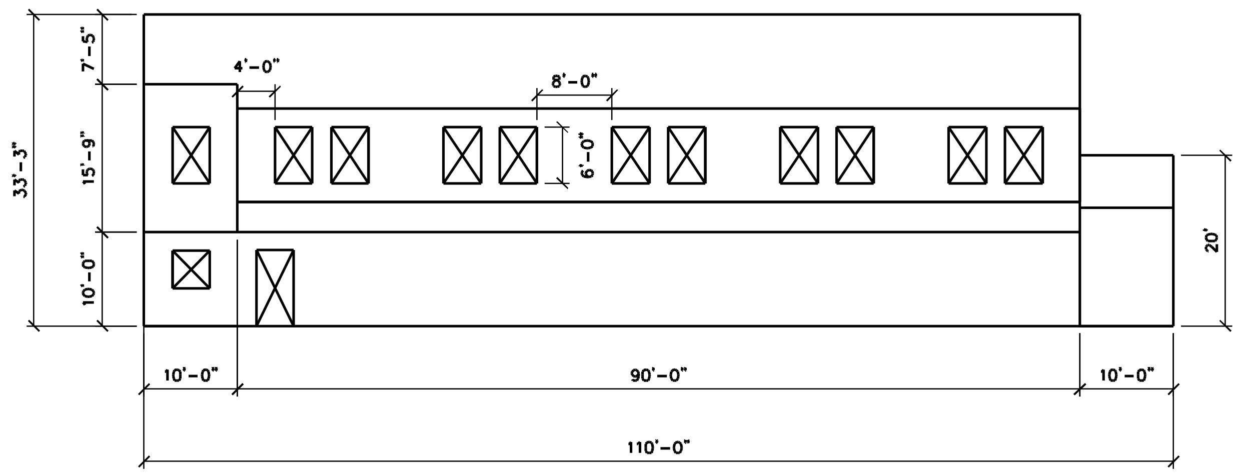


FRONT ELEVATION

1 foot = 0.305 m
1 inch = 25.4 mm

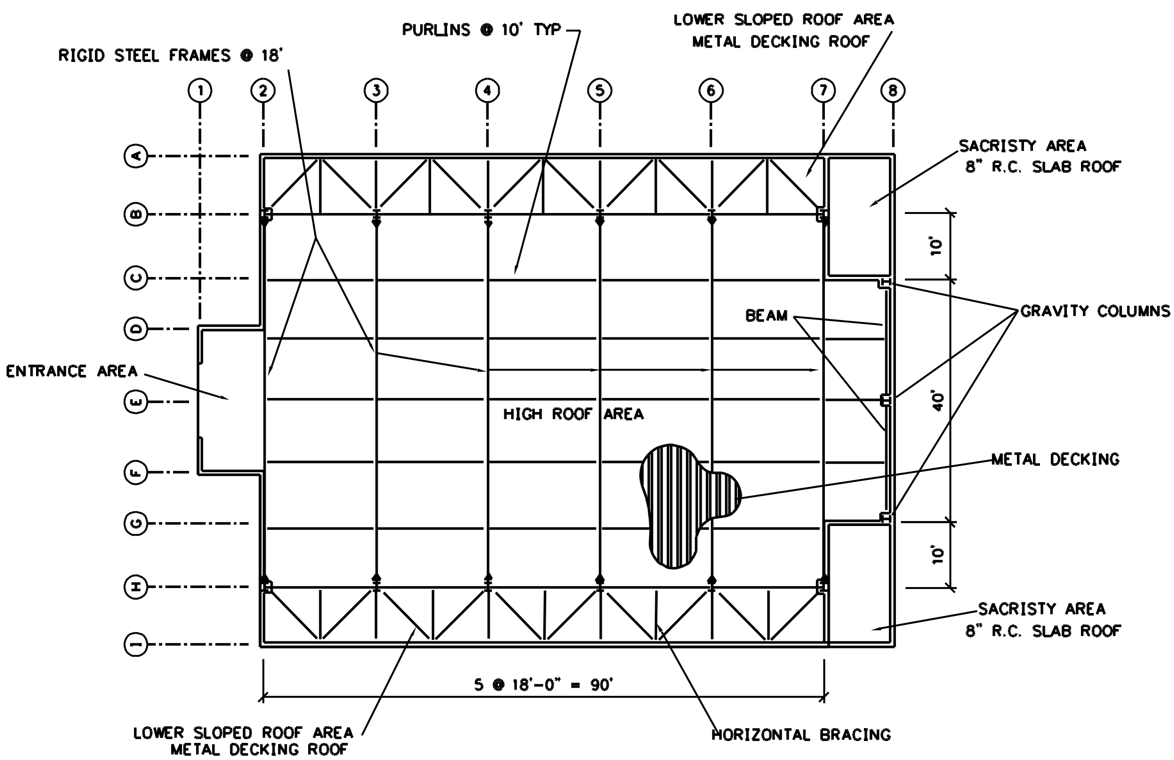


REAR ELEVATION

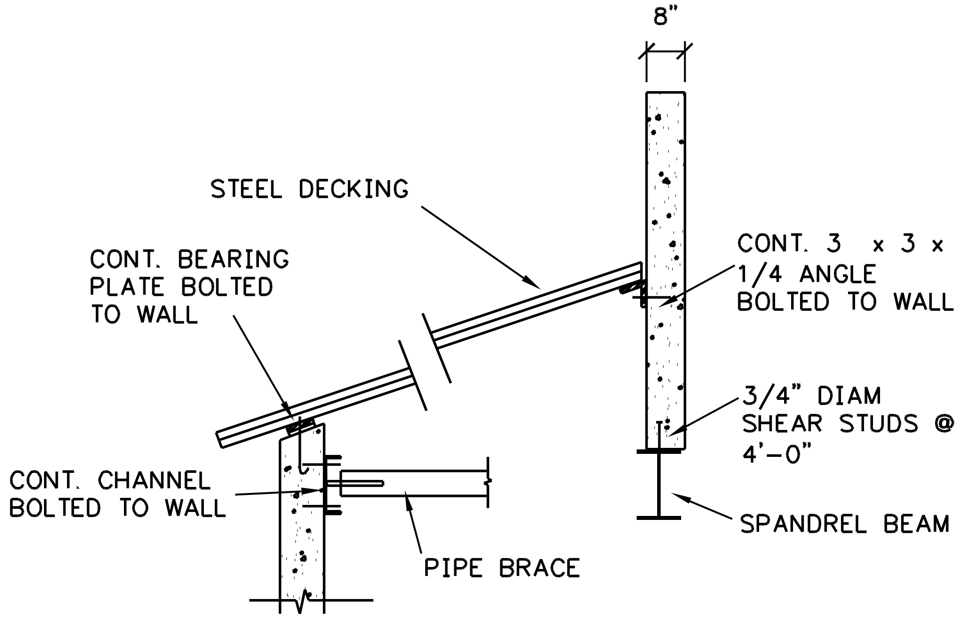


SIDE ELEVATION

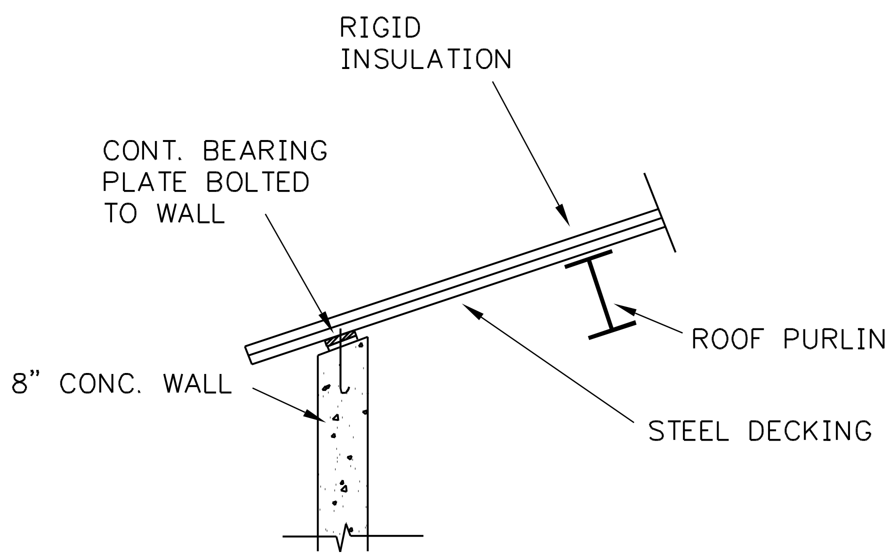
1 foot = 0.305 m
1 inch = 25.4 mm



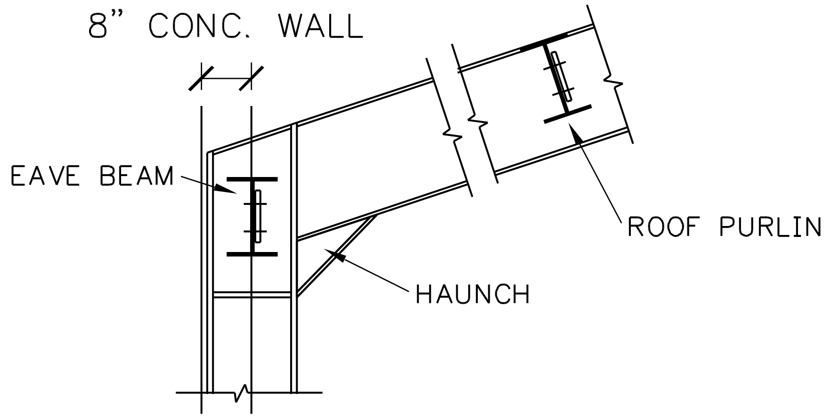
ROOF FRAMING



SECTION AT LOW ROOF



TYPICAL EAVE DETAIL



KNEE DETAIL FOR RIGID FRAME

c. Preliminary building design (Following steps in Table 4-5 for Life Safety). The preliminary design of the building follows the steps outlined in Table 4-5 for Performance Objective 1A. The design is then updated to meet the enhanced performance objectives laid out in Table 4-6 for Performance Objective 2A.

A-1 *Determine appropriate Seismic Use Group.* The structure falls into Seismic Use Group II.

A-2 *Determine Site Seismicity.* The site seismicity for this example from the MCE maps is:
$S_S = 1.50g$ $S_1 = 0.75g$.

A-3 *Determine Site Characteristics.* The soil for this example site is assumed to correspond to site class D.

A-4 *Determine Site Coefficients, F_a and F_v.* From Tables 3-2a and 3-2b for the given site seismicity and soil characteristics the site coefficients are:
$F_a = 1.0$ $F_v = 1.5$

A-5 *Determine adjusted MCE spectral response accelerations:*

$S_{MS} = F_a S_S = (1.0)(1.5g) = 1.50g$ Eq. 3-1

$S_{M1} = F_v S_1 = (1.5)(0.75g) = 1.13g$ Eq. 3-2

A-6 *Determine design spectral response accelerations:*

$S_{DS} = 2/3 S_{MS} = 2/3(1.5g) = 1.0g$ Eq. 3.3

$S_{D1} = 2/3 S_{M1} = 2/3(1.13g) = 0.75g$ Eq. 3.4

For regular structures, 5-stories or less in height, and having a period, T, of 0.5 seconds or less, the design spectral acceleration need not exceed:

$S_{DS} \le 1.5F_a = (1.5)(1.0) = 1.50g > 1.0g$ Eq. 3-5

$S_{D1} \le 0.6F_v = (0.6)(1.5) = 0.9g > 0.75g$ Eq. 3-6

A-7 *Select structural design category.* From Tables 4-2a and 4-2b for Seismic Use Group II and the design spectral response accelerations:
Seismic Design Category = D[a] Table 4-2a
Seismic Design Category = D[a] Table 4-2b
Footnote 'a' requires that structures with $S_1 \ge 0.75g$ be assigned to Seismic Design Category E.

A-8 *Select structural system.* The lateral system consists of a combination of special concrete shear walls and intermediate steel moment frames. In the transverse direction, the structure has concrete shear walls at the ends and moment frames in the interior. Longitudinally, concrete shear walls resist the lateral forces.

The upper roof diaphragm consists of metal decking that acts as a flexible diaphragm. The metal deck diaphragm spans are less than 2:1 in accordance with the limits for diaphragm span and depth set forth in Table 7-24. The lower roof diaphragm in the longitudinal direction consists of horizontal pipe bracing that transfers the shear from the suspended upper concrete shear walls to the exterior shear walls.

A-9 *Select R, W_o & C_d factors.*

Transverse (North-South): Building frame system with special reinforced concrete shear walls:

$R =$ 6
$\Omega_o =$ 2.5 Table 7-1
$C_d =$ 5

Longitudinal (East-West): Building frame system with special reinforced concrete shear walls:

$R =$ 6
$\Omega_o =$ 2.5 Table 7-1
$C_d =$ 5

A-10 *Determine preliminary member sizes for gravity load effects.*

Roof Purlins

- Assume simply supported, compression flange supported by roof deck along entire length
- Tributary width = spacing = 10' (3.05m)
- Strength reduction factor, $\phi = 0.9$ (AISC LRFD Section F1.2)

Loads:

Live loads: $L_r = 20R_1R_2$ ASCE 7-95 Eq. 4-2

$A_t = (10')(18') = 180$ ft.^{2} (16.7 m^2)

$R_1 = 1$ ($A_t < 200$ ft.^{2} or 18.6 m^2)

$R_2 = 1$ (Rise of roof = 4" per foot or 3.33 cm per 10 cm)

$L_r = (20$ psf$)(1)(1) = 20$ psf (958 N/m^2)

Dead loads:	Finish	1.0 psf
	Metal Decking	2.0 psf
	Purlins (self wt.)	1.5 psf
	Ceiling / Covering	1.0 psf
	Misc.	3.0 psf
	Total:	8.5 psf (407 N/m^2)
	Adjust for slope:	(8.5)(12.65/12) = 8.96 plf (429 N/m^2)

Load Combination: 1.2 D + 1.6 Lr

$w_u = (10')[(1.2)(8.96\text{psf}) + (1.6)(20\text{psf})] = 428$ plf (6.25 KN/m)

$Z_{req} = M_u / \phi F_y$,

$M_u = w_u L^2/8 = (428 \text{ plf})(18')^2/8 = 17.3$ kipft (23.46 KNm)

$Z_{req} = (17.3)(12)/(0.9)(36) = 6.41$ in.^{3} (105 cm^3)

Try C8x11.5, $Z_x = 9.55$ in.^{3} (156 cm^3) > 6.41 in.^{3}

Check shear;

$V_u = (428 \text{ plf})(18')/2 = 3.9$ kips (17.35 KN)

$h/t_w = 6.125/.22 = 27.8 < 418/(36)^{1/2} = 70$, use AISC LRFD Eq. F2-1

$\phi_v V_n = 0.6F_{yw}A_w = 0.6(36\text{ksi})(0.22")(8.0") = 38.0$ kip (169 KN) > 3.9 kips (17.35KN), O.K.

Beams along grid line 8

- Assume simply supported, compression flange supported by roof deck
- Tributary width = 5' (1.53m)

Loads:

Live Loads: $L_r = 20R_1R_2$ ASCE 7-95 Eq. 4-2

$A_t = (5')(20') = 100$ ft.^{2} (9.3m^2)

$R_1 = 1$ ($A_t < 200$ ft.^{2})

$R_2 = 1$ (Rise of roof = 4" per foot)

$L_r = (20$ psf$)(1)(1) = 20$ psf (958 N/m^2)

Dead Loads: (10.5)(12.65/12); Same as for purlins but add 2 psf (96 N/m^2) for self-weight

= 11.07 psf (530 N/m^2)

Load Combination: 1.2D + 1.6Lr

$w_u = (5)[1.2(11.07)+1.6(20)] = 226$ plf (3.30 KN/m)

$M_u = (226)(20)^2/8 = 11.3$ kipft (15.32 KNm)

$Z_{req} = (11.3)(12)/(0.9)(36) = 4.2$ in.^{3} (68.8 cm^3)

Try W12x14, $Z_x = 17.4$ in.^{3} (285 cm^3) > 4.2 in.^{3} (68.8 cm^3)

Check shear;

$V_u = (428 \text{ plf})(18')/2 = 3.9$ kips (17.3 KN)

$h/t_w = 54.3 < 418/(36)^{1/2} = 70$, use AISC LRFD Eq. F2-1

$\phi_v v_n = 0.6F_{yw}A_w = 0.6(36ksi)(0.20")(11.91") = 51.5$ kip (229 KN) > 3.9 kips (17.3 KN), O.K

Columns C8, E8 and G8

- Columns are designed for gravity loads only
- Assume columns are pin-pin connected, K=1.0
- Use L = 33.33' (10.17m) for all columns (conservative)
- Tributary area = 100 ft.^{2} (9.3 m^2)

  Loads:

  Live Loads: 20 psf (958 N/m^2)

  Dead Loads: 11.07 psf (530 N/m^2); Same as for beams along grid line 8

  Load Combination: 1.2D + 1.6Lr

  $P_u = (100)[(1.2)(11.07)+(1.6)(20)] = 4.53$ kips (20.15 KN)

  Try W10x33, A = 9.71 in.^{2} (62.6 cm^2), r_y = 1.94 in. (4.93 cm)

$$\lambda_c = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{(1.0)(33.3)(12)}{1.94\pi}\sqrt{\frac{36}{29000}} = 2.31 > 1.5$$ AISC LRFD Eq. E2-4

$$F_{cr} = \left(\frac{0.866}{\lambda_c^2}\right)F_y = \left(\frac{0.866}{2.31^2}\right)36 = 5.84\text{ksi}$$ AISC LRFD Eq. E2-3

$$\phi_c P_n = \phi_c A_g F_{cr} = (0.85)(9.71)(5.84) = 48.2\text{kips}\ (214\text{KN}) > 4.53\text{k}\ (20.2\text{ KN})$$ AISC LRFD Eq. E2-1

Beams along grid lines B & H

These beams support the discontinuous 8" (20.32 cm) thick concrete window walls. It is assumed that the beams carry no live load. It is further assumed that the beams are completely supported by the concrete walls for torsion. Therefore, torsion is not considered.

Loads:

Dead Loads: (from concrete wall above)

Weight of wall supported by beams = (unit weight)(wall height) = (8/12)(150 pcf)(13.25') = 1325 plf

$w_u = 1.4\ D = 1.4(1325$ plf) = 1855 plf (27.1 KN/m) (ASCE 7-95 Sec. 2.3.2)
$M_u = (1855)(18)^2/8 = 75.13$ kipft (102 KNm)
$Z_{req} = (75.13)(12)/(0.9)(36) = 28$ in.^{3} (459 cm^3)

Try W14 x 38, Z_x = 61.5 in.^{3} > 28 in.^{3}

Check shear;
V_u = (1855 plf)(18')/2 = 16.7 kips (74.3 KN)
$h/t_w = 39.6 < 418/(36)^{1/2} = 70$, use AISC LRFD Eq. F2-1
$\phi_v v_n = 0.6F_{yw}A_w = 0.6(36ksi)(0.31")(14.10") = 96.4$ kip (428.8 KN) > 16.7 kips (74.3 KN) O.K

Moment Frames

For the preliminary gravity design it was assumed that $I_c / I_b = 1.75$ & $A_c / A_b = 1.75$ to ensure strong col / weak beam. Trial sections were obtained and the analysis repeated with the assumed sections.

– Frames are spaced at 18' (5.49m)
– Dead load from roof = 12.1 psf (579 N/m) on horizontal projection (See step B-2)

Columns: Loads:
The columns support the loads from the upper roof as well as the beams along grid lines B & H (one on each side of column)

Dead Loads:
12.1 psf (roof dead load from step B-2)
Beam Reaction = 2*(wL/2) = (2)(1325 plf)(18')/2 = 23.9 k (106 KN)

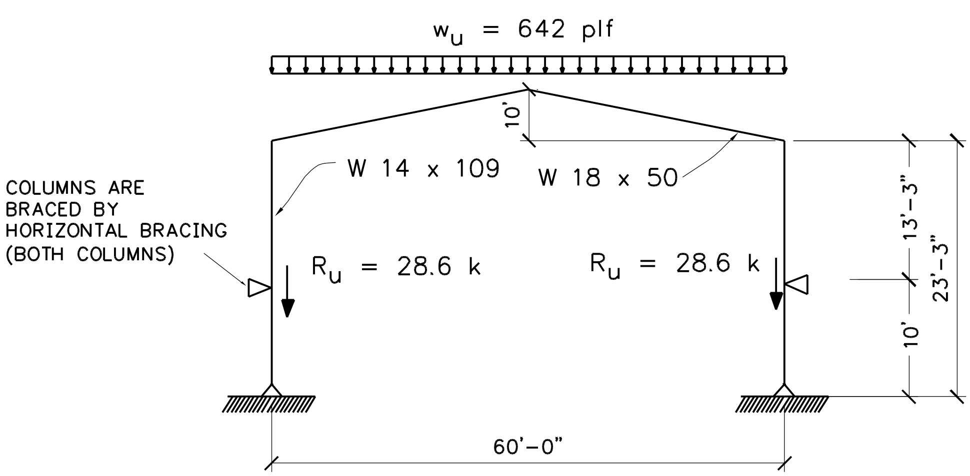
Live Loads: Live loads: $L_r = 20R_1R_2$ ASCE 7-95 Eq. 4-2
A_t = (18')(30') = 540 ft.^{2} (50.22 m^2)
$R_1 = 1.2 – 0.001A_t = 1.2 – 0.001(540) = 0.66$
R_2 = 1 (Rise of roof = 4" per foot)
L_r = (20 psf)(0.66)(1) = 13.2 psf (193 N/m^2)

Beams: (Assume beam is one member for design, Length = 60' or 18.3m)
Loads;
Dead Loads: (12.1 psf)(18') = 218 plf (3.18 KN/m)

Live Loads: The tributary area of the beams (18')(60') = 1080 ft.^{2} (100.4m^2)
The trib. area of the beams is greater than that of the columns (540 ft.^{2} or 50.2 m^2)
A higher live load reduction could be used, however use the same live loads that the columns to be conservative
L_r = (20 psf)(0.66)(1) = 13.2 psf = (13.2psf)(18') = 238 plf (3.47 KN/m)
w_u = 1.2(218) + 1.6(238) = 642 plf (9.37 KN/m)

1 plf = 14.59 N/m
1 kip = 4.448 KN
1 foot = 0.305 m
1 inch = 25.4 mm

Input for elastic analysis



Beam Design

The beams are laterally braced by the purlins, $L_b = 10'$ (3.1m)

- Results from elastic analysis:

  Maximum moment at end of beam = 125.3 kft (170 KNm)
  Maximum axial force at end of beam = 21.25 k (94.5 KN)

Design member as beam-column
- Try a W 18 x for trial size
- $KL_x = 1.2(31.6) = 38$ ft., $Kl_y = 1.0(10') = 10$
- $L_b = 10'$ (spacing of purlins)
- $M_y = 0$ (no lateral forces on beam)

Try W 18 x 50 (A = 14.7 in.^{2}, $r_x = 7.38$ in., $r_y = 1.65$ in., $L_p = 6.9'$, BF = 7.31, $\phi_b M_p = 273$ kft)
$(KL/r)x = (38')(12")/(7.38") = 61.8$
$(KL/r)y = (10')(12")/(1.65") = 72.7$
$\phi_c F_{cr} = 23.17$ ksi (160 N/mm^2) (AISC Table 3-36)
$\phi P_n = A\phi_c F_{cr} = (14.7)(23.17) = 340$ k (1512 KN)
$\phi_b M_{nx} = C_b[\phi_b M_{nx} - BF(L_b - L_p)] = 1.0(273\text{kft} - 7.31(10 - 6.9)) = 250\text{kft}$ (339 KNm) (AISC LRFD Chap 4)
$P_u / \phi P_n = 21.25 / 340 = 0.06 < 0.2$, use interaction equation H1-1b

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}}\right) \le 1 = \frac{21.25}{2(340)} + \left(\frac{125.28}{250}\right) = 0.53 < 1, \text{OK}$$

By inspection assume that beam is OK in shear.
The beam is slightly over designed in anticipation of the higher stresses due to the addition of the lateral loads to the frames.
Use W 18 x 50 as a trial beam size.

Column Design

The interaction of axial and moment forces are check at the top of the column (highest moment) and at the horizontal support point (highest axial load)

- Results from elastic analysis:

  Moment at top of column = 125.3 kft = 1504 kipin (170 KNm)
  Axial force at top of column = 20.31 k (90.3KN)

  Moment at horizontal pin support (at horizontal bracing) = 81.9 kft = 983 kipin (111 KNm)
  Axial force at pin support = 48.99 k (218 KN)

Design member as beam-column

Try a W 14 x 109 for trial size to ensure strong column / weak beam
W 14 x 109 (A = 32.0 in.^{2}, $r_x = 6.22$ in., $r_y = 3.73$ in., $L_p = 15.5'$, $Z_x = 192$ in.^{3})

- Top of column:
  The upper portion of the column (above the horizontal bracing level) is braced in both directions from the concrete window walls (B1-B2 & H1-H2). However, an unbraced column length of 13.3' will be used to determine the axial capacity (based on $K_x L_x / r_x$) to be conservative. It is assumed that the compression flange of the columns is continuously supported by the concrete walls.

Determine K from alignment chart for effective length of columns in continuous frames (AISC LRFD Fig C-C2.2)
Assume that the column is pinned at the lateral support (G=10).

At top of column, $G = \dfrac{\sum(I_c / L_c)}{\sum(I_g / L_g)} = \dfrac{(1240/(13.3)}{(800/31.6)} = 3.68$, $K \approx 2.4$ (sidesway uninhibited)

$(KL/r)x = (2.4)(13.3')(12)/(6.22") = 61.58$
$\phi_c F_{cr} = 25.06$ ksi (173 N/mm^2) (AISC LRFD Table 3-36)
$\phi_c P_n = 25.06$ ksi (32 in.^{2}) = 802 kips (3567 KN)

$M_{ux} = B_{1x}M_{ntx}+B_2xM_{ltx}$; for this preliminary design the moment magnification factors are assumed to be unity. Therefore, M_{ux} = moment at top of column = 125.3 kft (170 KNm)

$\phi_b M_{nx} = \phi_b Z_x F_y = (0.9)(192)(36)/(12) = 518$ kft (702 KNm); (see AISC LRFD Chapter F)

$P_u / \phi P_n = 20.31 / 802 = 0.025 < 0.2$, use interaction equation H1-1b

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}}\right) \le 1 = \frac{20.31}{2(802)} + \left(\frac{125.3}{518}\right) = 0.25 < 1, \text{ OK}$$

- Bottom of column:
  The lower portion of the column (below the horizontal bracing level) is assumed to be pinned at both the footing support and the horizontal bracing for determining the second order effects.

K = 1.0 in both x and y directions (pin support)

$(KL/r)_x = (1.0)(10')(12)/(6.22") = 19.3$
$(KL/r)_y = (1.0)(10')(12)/(3.73") = 32.2$ (governs)
$\phi_c F_{cr} = 29$ ksi (200 N/mm^2) (AISC LRFD Table 3-36)
$\phi_c P_n = 29$ ksi (32 in.^{2}) = 927 kips (4123 KN)

$M_{ux} = B_{1x}M_{ntx}+B_2xM_{ltx}$; for this preliminary design the moment magnification factors are assumed to be unity. Therefore, M_{ux} = moment at bracing support = 125.3 kft (170 KNm)

$\phi_b M_{nx} = \phi_b Z_x F_y = (0.9)(192)(36)/(12) = 518$ kft (702 KNm), since $L_b < L_p$ (10 < 15.5) (see AISC LRFD Chapter F)

$P_u / \phi P_n = 48.99/ 927 = 0.05 < 0.2$, use interaction equation H1-1b

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}}\right) \le 1 = \frac{48.99}{2(927)} + \left(\frac{81.9}{518}\right) = 0.18 < 1, \text{ OK}$$

The column is over designed in anticipation of the higher stresses due to the addition of the lateral loads to the frames and to ensure strong beam / weak column.
Use W 14 x 109 as a trial column size.

B-1 *Calculate fundamental period, T*

$T_a = C_T h_n^{3/4}$ where C_T = 0.020 and h_n = 28.25 ft. (ave. roof ht.) (FEMA 302 Eq. 5.3.3.1-1)

$T_a = (0.020)(28.25)^{3/4} = 0.25$ seconds for both the transverse and longitudinal directions.

B-2 *Determine the dead load, W*

Sloped Upper Roof Level*

Finish	1.0 Psf
20 Gage Metal Deck	2.0 Psf
Roof Purlins	1.5 Psf
Rigid Frame Beams	2.0 Psf
Rigid Frame Columns	1.0 Psf
Ceiling / Covering	1.0 Psf
Misc. (Mech., Elec. & Framing)	3.0 Psf
Total:	11.5 Psf

Sloped Lower Roof Level*

Built-Up Roofing	5.0 psf
2" Rigid Insulation	3.0 psf
20 Gage Metal Deck	2.0 psf
Horizontal Bracing	2.0 psf
Ceiling / Covering	1.0 psf
Misc. (Mech., Elec.& Framing)	3.0 psf
Total:	16.0 psf

Roof at Sacristy (Horizontal)

Finish	1.0 Psf
Concrete Slab (Assume 4" Thick)	50.0 Psf
Ceiling / Covering	1.0 Psf
Misc. (Mech, Elec.)	1.0 Psf
Total:	53.0 Psf

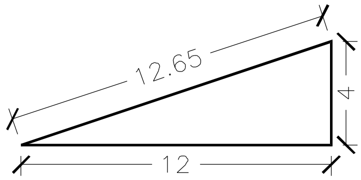
Sloped Roof at Entrance*

Built-Up Roofing	5.0 psf
2" Rigid Insulation	3.0 psf
20 Gage Metal Deck	2.0 psf
Steel Beams	1.0 psf
Ceiling / Covering	1.0 psf
Misc. (Mech, Elec. & Framing)	1.0 psf
Total:	13.0 psf

Reinforced Concrete Shear Walls

Normal Weight Concrete (8" Thick) 100 Psf

*Note: The weights of the sloped roofs are calculated based on the sloped area. In order to calculate the weight based on the horizontally projected area the loading on the sloped roof must be adjusted.



Upper roof level: (11.5)(12.65/12) = 12.12 psf
Sloped lower roof level: (16.0)(12.65/12) = 16.87 psf
Roof at entrance: (13.0)(12.65/12) = 13.70 psf

1 psf = 47.88 N/m^2

BUILDING SEISMIC WEIGHTS

UPPER ROOF LEVEL TRIBUTARY SEISMIC WEIGHTS (ROOF & TRIBUTARY WALLS)

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)	
UPPER ROOF							
Roof (grid 2 - 7)	62.0	90.0	100.0	5580.0	12.1	67.6	
Roof (grid 7-8)	10.0	42.0	100.0	420.0	12.1	5.1	
TRANSVERSE WALLS							
Wall 8C-8G	---	---	---	650.0	100.0	65.0	
Wall 7A-7C	---	---	---	66.0	100.0	6.6	
Wall 7G-7I	---	---	---	66.0	100.0	6.6	
Wall 2B-2H	---	---	---	800.0	100.0	80.0	
LONGITUDINAL WALLS							
Wall B2-B7	6.5	90.0	65.0	380.3	100.0	38.0	
Wall H2-H7	6.5	90.0	65.0	380.3	100.0	38.0	
Wall C7-C8	8.0	10.0	75.0	60.0	100.0	6.0	
Wall G7-G8	8.0	10.0	75.0	60.0	100.0	6.0	
					TOTAL	319.0	1419 KN

LOWER SLOPED ROOF LEVEL TRIBUTARY SEISMIC WEIGHTS (ROOF & TRIBUTARY WALLS)

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)	
LOWER ROOF							
Roof A2-B7	10.0	90.0	100.0	900.0	16.9	15.2	
Roof H2-I7	10.0	90.0	100.0	900.0	16.9	15.2	
TRANSVERSE WALLS							
Wall 7A-7B	---	---	---	62.0	100.0	6.2	
Wall 7H-7I	---	---	---	62.0	100.0	6.2	
Wall 2A-2B	---	---	---	62.0	100.0	6.2	
Wall 2H-2I	---	---	---	62.0	100.0	6.2	
LONGITUDINAL WALLS							
Wall B2-B7	6.5	90.0	65.0	380.3	100.0	38.0	
Wall H2-H7	6.5	90.0	65.0	380.3	100.0	38.0	
Wall A2-A7	5.0	90.0	95.0	427.5	100.0	42.8	
Wall I2-I7	5.0	90.0	95.0	427.5	100.0	42.8	
					TOTAL	216.7	964 KN

LOWER SACRISTRY TRIBUTARY SEISMIC WEIGHTS

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)	
LOWER ROOF							
Roof A7-C8	10.0	19.7	100.0	196.7	53.0	10.4	
Roof G7-I8	10.0	19.7	100.0	196.7	53.0	10.4	
TRANSVERSE WALLS							
Wall 8A-8C	5.0	20.0	80.0	80.0	100.0	8.0	
Wall 8G-8I	5.0	20.0	80.0	80.0	100.0	8.0	
LONGITUDINAL WALLS							
Wall A7-A8	5.0	10.0	75.0	37.5	100.0	3.8	
Wall I7-I8	5.0	10.0	75.0	37.5	100.0	3.8	
Wall C7C8	13.0	10.0	75.0	97.5	100.0	9.8	
Wall G7-G8	13.0	10.0	75.0	97.5	100.0	9.8	
					TOTAL	63.9	284 KN

LOWER SLOPED ROOF @ ENTRANCE TRIBUTARY SEISMIC WEIGHTS

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)	
LOWER ROOF							
Roof D1-F2	10.0	24.0	100.0	240.0	13.7	3.3	
TRANSVERSE WALLS							
Wall D1-F1	---	---	100.0	160.0	100.0	16.0	
LONGITUDINAL WALLS							
Wall D1-D2	8.0	10.0	100.0	80.0	100.0	8.0	
Wall F1-F2	8.0	10.0	100.0	80.0	100.0	8.0	
					TOTAL	35.3	157 KN

Total Seismic Weight = 634.8 Kips (2824 KN)

B-3 *Calculate Base Shear*

$V = C_sW$ FEMA 302 Eq. 5.3.2

$C_s = S_{DS}/R$, but need not exceed: Eq. 3-7

$C_s = S_{D1}/TR$, but shall not be less than: Eq. 3-8

$C_s = 0.044S_{DS}$ Eq. 3-9

Transverse Direction:

$C_s = (1.0)/6 = 0.167$

$C_s = (0.75)/(0.25)(6) = 0.5 > 0.167$

$C_s = 0.044(1.0) = 0.044 < 0.167$

$V = C_sW = (0.167)(634.8 \text{ kips}) =$ 106 kips (471 KN)

Longitudinal Direction:

$C_s = (1.0)/6 = 0.167$

$C_s = (0.75)/(0.25)(6) = 0.5 > 0.167$

$C_s = 0.044(1.0) = 0.044 < 0.167$

$V = C_sW = (0.167)(634.8 \text{ kips}) =$ 106 kips (471 KN)

B-4 *Calculate vertical distribution of seismic forces*

The building is analyzed as a one-story structure. The seismic forces to the separate roof areas are determined by multiplying the tributary weights by the seismic coefficient C_s.

<u>Transverse Direction:</u> $C_s =$ 0.167

$W_{roof} =$	318.99 Kips	$V_{roof} =$	53.2 kips (237 KN)
$W_{lowroof} =$	108.36 Kips each	$V_{lowroof} =$	18.1 kips each (81 KN)
$W_{sacr} =$	31.93 Kips each	$V_{sacr} =$	5.3 kips each (23.6 KN)
$W_{entrance} =$	35.29 Kips	$V_{entrance} =$	5.9 kips (26.2 KN)

<u>Longitudinal Direction:</u> $C_s =$ 0.167

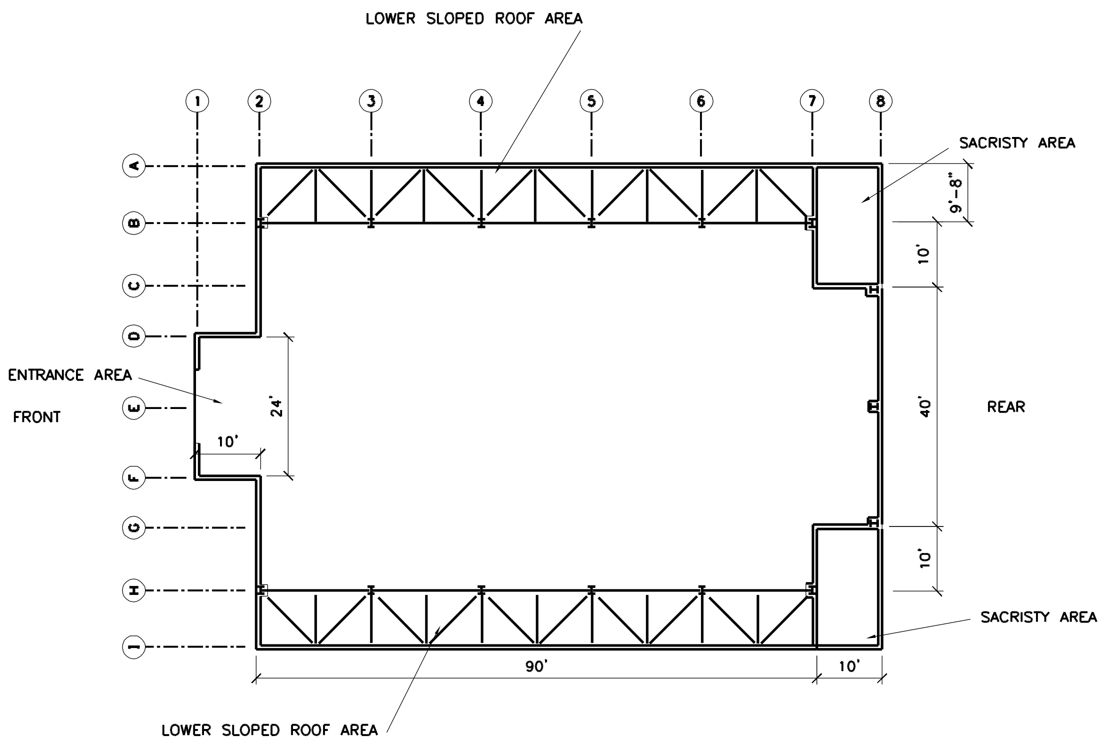
$W_{roof} =$	318.99 Kips	$V_{roof} =$	53.2 kips (237 KN)
$W_{lowroof} =$	108.36 Kips each	$V_{lowroof} =$	18.1 kips each (81 KN)
$W_{sacr} =$	31.93 Kips each	$V_{sacr} =$	5.3 kips each (23.6 KN)
$W_{entrance} =$	35.29 Kips	$V_{entrance} =$	5.9 kips (26.2 KN)

B-5 *Perform Static Analysis*

The seismic forces from the upper sloped roof and entrance are resisted by the vertical elements based on tributary areas due to flexible diaphragm action. The seismic forces tributary to the sacristies and lower sloped roof areas are resisted by the vertical elements based on relative rigidity due to rigid diaphragm action.

<u>Seismic forces to vertical resisting elements from upper sloped roof diaphragm</u>

<u>Transverse direction:</u> Seismic forces tributary to the sloped upper roof in the transverse direction are resisted by the steel moment frames and the concrete shear walls based on tributary areas. It is assumed that the frames along grid lines 2 and 7 resist no loads due to their low stiffness as compared to the shear walls along the same grid lines. The diaphragm forces are determined from the weight of the roof and tributary normal walls. It is assumed that the diaphragm acts as a simply supported beam element between the frames and shear walls.



FLOOR PLAN

1 inch = 25.4mm
1 foot = 0.305m

Weight of roof and normal walls between grid lines 2 and 7

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)
UPPER ROOF						
Roof (grid 2 - 7)	62.0	90.0	100.0	5580.0	12.1	67.6
LONGITUDINAL WALLS						
Wall B2-B7	6.5	90.0	65.0	380.3	100.0	38.0
Wall H2-H7	6.5	90.0	65.0	380.3	100.0	38.0
					TOTAL	143.7

(639 K)N

Total weight = 143.7 kips Seismic coefficient, $C_s = 0.167$
Seismic force = C_sW = (0.167)(143.7) = 24 kips (107 KN)
Equivalent Running Load $w_{2\text{-}7}$ = 24 k / 90 ft. = 267 plf (3.90 KN/m)

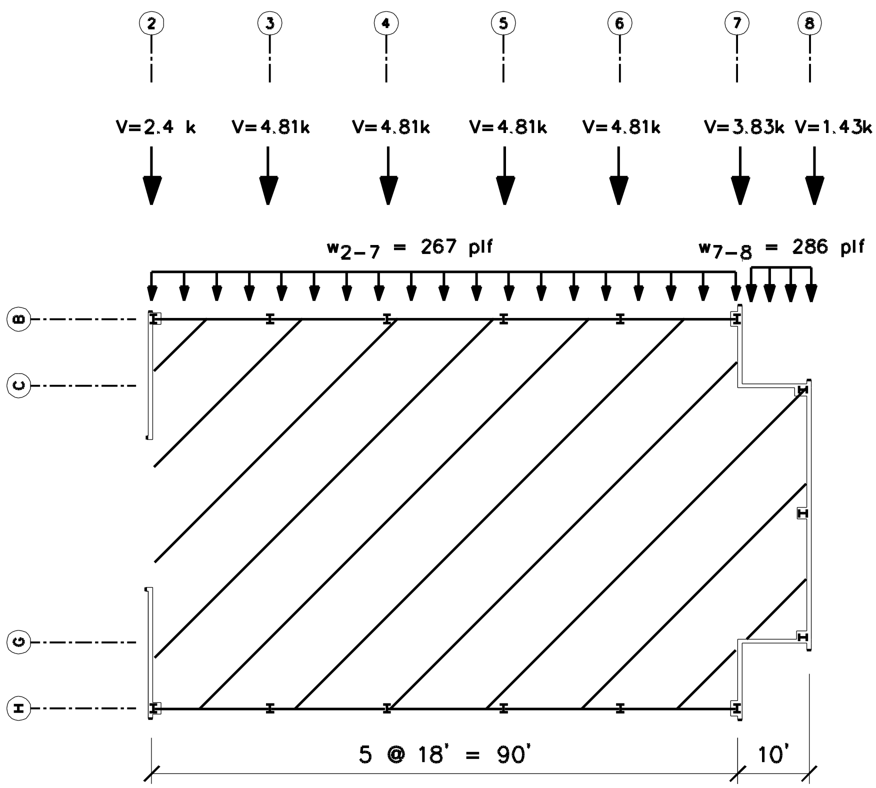
Weight of roof and normal walls between grid lines 7 and 8

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)
UPPER ROOF						
Roof (grid 7-8)	10.0	42.0	100.0	420.0	12.1	5.1
LONGITUDINAL WALLS						
Wall C7-C8	8.0	10.0	75.0	60.0	100.0	6.0
Wall G7-G8	8.0	10.0	75.0	60.0	100.0	6.0
					TOTAL	17.1

76.1 KN

Total weight = 17.1 kips Seismic coefficient, C_s = 0.167
Seismic force = C_sW = (0.167)(17.1) = 2.86 kips (12.7 KN)
Equivalent Running Load $w_{7\text{-}8}$ = 2.86 k / 10 ft. = 286 plf (4.17 KN/m)

Shear force to wall line 2 from upper roof diaphragm:
V = $w_{2\text{-}7}$(9') = (267 plf)(9') = 2.40 k (10.7 KN)
Shear force to each rigid frame (grid lines 3, 4, 5 and 6)
V = $w_{2\text{-}7}$(18') = (267 plf)(18') = 4.81 k (21.4 KN)
Shear force to wall line 7
V = $w_{2\text{-}7}$(9') + $w_{7\text{-}8}$(5') = (267)(9) + (286)(5) = 3.83 k (17.0 KN)
Shear force to wall line 8 V = $w_{7\text{-}8}$(5') = (286)(5) =1.43 k (6.36 KN)



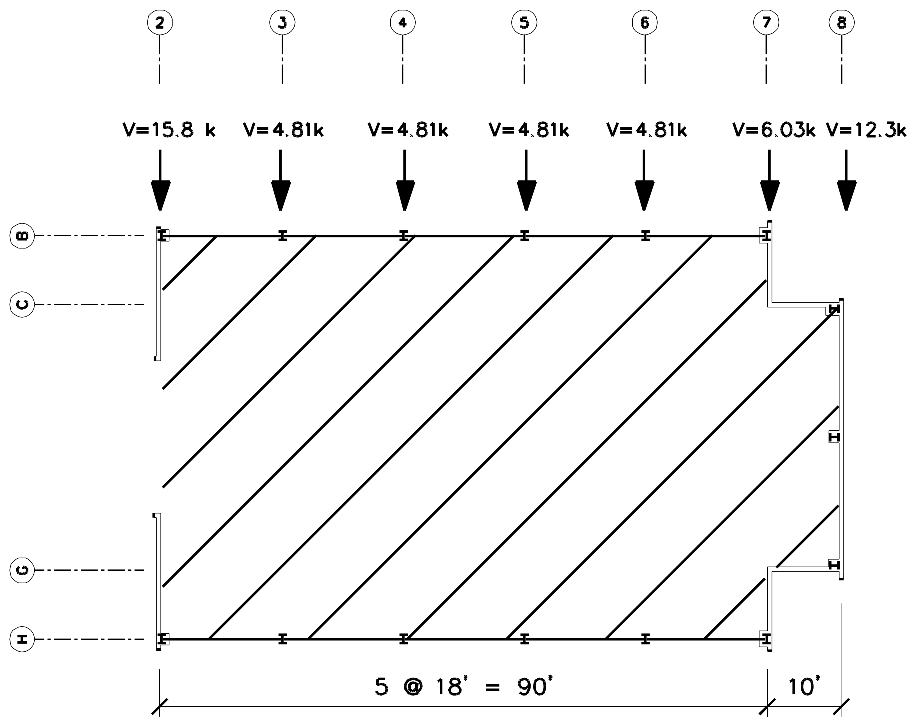
1 kip = 4.448 KN
1 foot = 0.305m
1 plf = 14.59 N/m

The self-weight inertial effects of the shear walls due to the weight tributary to the upper sloped roof must now be added to the shears determined for shear wall lines 2, 7 and 8.

– Weight of wall line 2 tributary to the upper roof = 80 kips
  Self-weight inertial force = C_sW = (0.167)(80) = 13.4 kips
  Total shear to wall line 2 tributary to upper roof diaphragm = (2.4 k) + (13.4 k) = 15.8 k (70.3 KN)

– Weight of wall line 7 tributary to the upper roof = (6.6) + (6.6) = 13.2 k (58.7 KN)
  Self-weight inertial force = C_sW = (0.167)(13.2) = 2.2 k (9.8 KN)
  Total shear to wall line 7 tributary to upper roof diaphragm = (3.83 k) + (2.2 k) = 6.03 k (26.8 KN)

– Weight of wall line 8 tributary to the upper roof = 65 k
  Self-weight inertial force = CW = (0.167)(65 k) = 10.9 k
  Total shear to wall line 8 tributary to upper roof diaphragm = (1.43 k) + (10.9 k) = 12.3 k (54.7 KN)



1 kip = 4.448 KN
1 foot = 0.305m

Longitudinal Direction: Seismic forces tributary to the upper roof in the longitudinal direction are resisted by the concrete shear walls based on tributary areas. Shear wall lines B2-B7 & H2-H7 each resist ½ of the shear from the upper roof between lines 2 and 7. Shear wall lines C7-C8 and G7-G8 each resist ½ of the shear from the upper roof between lines 7 and 8. The shear force associated with normal wall line 8 is assumed to be resisted by shear wall lines C and G, while the shear force associated with normal wall line 7 is assumed to be resisted by shear wall lines B & H. It is assumed that the diaphragm acts as a simply supported beam element between the shear walls.

Weight of roof and normal walls between grid lines 2 and 7

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)	
UPPER ROOF							
Roof (grid 2 - 7)	62.0	90.0	100.0	5580.0	12.1	67.6	
TRANSVERSE WALLS							
Wall 7A-7C	---	---	---	66.0	100.0	6.6	
Wall 7G-7I	---	---	---	66.0	100.0	6.6	
Wall 2B-2H	---	---	---	800.0	100.0	80.0	
					TOTAL	160.8	715 KN

Total weight = 160.8 kips Seismic coefficient, C = 0.167
Seismic force = C_sW = (0.167)(160.8) = 26.85 kips
Equivalent Running Load $w_{b\text{-}h}$ = 26.85 k / 60 ft. = 448 plf

Weight of roof and normal walls between grid lines 7 and 8

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)	
UPPER ROOF							
Roof (grid 7-8)	10.0	42.0	100.0	420.0	12.1	5.1	
TRANSVERSE WALLS							
Wall 8C-8G	---	---	---	650.0	100.0	65.0	
					TOTAL	70.1	312 KN

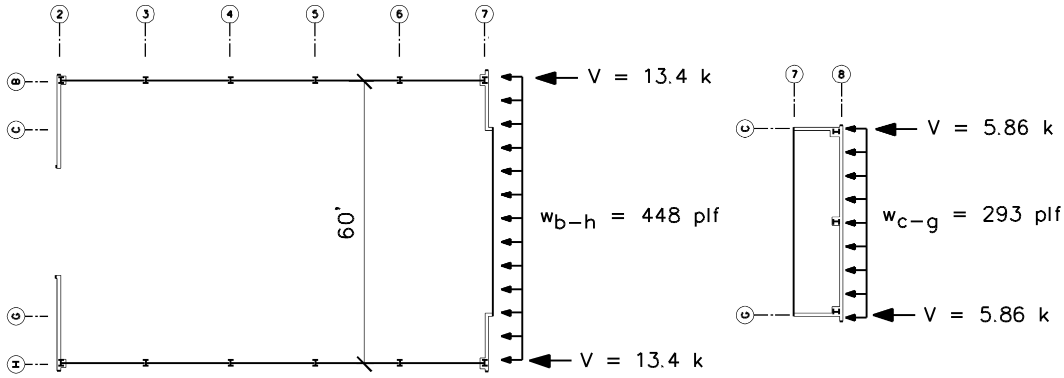
Total weight = 70.1 kips Seismic coefficient, C = 0.167
Seismic force = C_sW = (0.167)(70.1) = 11.71 kips (52.1 KN)
Equivalent Running Load $w_{c\text{-}g}$ = 11.71 k / 40 ft. = 293 plf (4.27 KN/m)

– Shear force to wall line B from upper roof diaphragm
  V = $w_{b\text{-}h}$(30’) = (448)(30) = 13.4 kips (59.6 KN)

– Shear force to wall line H (same as wall line B due to symmetry)
  V = 13.4 kips (59.6 KN)

– Shear force to wall line C
  V = $w_{c\text{-}g}$(20’) = (293)(20) = 5.86 kips (26.1 KN)

– Shear force to wall line G (same as wall line C due to symmetry)
  V = 5.86 kips (21.6 KN)



1 kip = 4.448 KN
1 plf = 14.59 N/m

The self-weight inertial effects of the shear walls due to the weight tributary to the upper roof must now be added to the shears determined for shear wall lines B, C, G and H.

– Weight of wall line B tributary to the upper roof = 38 kips
  Self-weight inertial force = C_sW = (0.167)(38) = 6.35 kips
  Total shear to wall line B tributary to upper roof diaphragm = (13.4 k) + (6.35 k) = 19.8 k (88.1 KN)

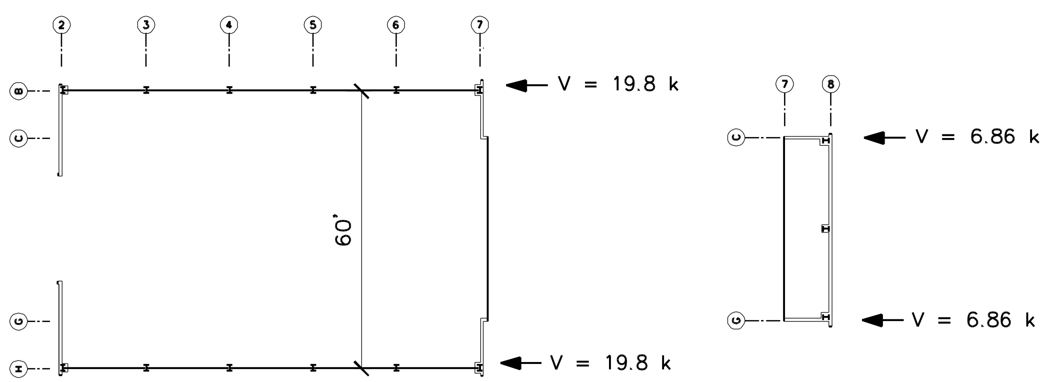
– Wall line H same as line B, Total shear = 19.8 k (88.1 KN)

  The shear from wall lines B & H is transferred through the horizontal bracing to shear wall lines A & I, respectively.

– Weight of wall line C tributary to the upper roof = 6 k
  Self-weight inertial force = C_sW = (0.167)(6) = 1.0 k
  Total shear to wall line C tributary to upper roof diaphragm = (5.86 k) + (1.0 k) = 6.86 k (30.5 KN)

– Wall line G same as line C, Total shear = 6.86 (30.5 KN)

TOTAL SHEAR TO VERTICAL ELEMENTS TRIBUTARY TO UPPER ROOF



Chord forces:
The upper roof diaphragm is analyzed as two subdiaphragms (one for grid lines 2-7 and one for grid lines 7-8).

Transverse direction:
The spans act as simply supported elements between the moment frames and concrete shear walls.

$w_{2\text{-}7}$ = 267 plf (3.90 KN/m)
M = $wL^2/8$ = $(0.267)(18\ ft)^2$ / 8 = 10.8 kipft (14.64 KNm)
T = M / d = (10.8 / 60 ft) = 0.18 kips (801N)

$w_{7\text{-}8}$ = 286 plf (4.17 KN/m)
M = $wL^2/8$ = $(0.286)(10\ ft)^2$ / 8 = 3.58 kipft (4.85 KNm)
T = M / d = (3.58 / 40 ft) = 0.09 kips (0.40 KN)

Longitudinal direction:
The spans act as simply supported elements between the concrete shear walls.

$w_{2\text{-}7}$ = 448 plf (6.54 KN/m)
M = $wL^2/8$ = $(0.448)(60\ ft)^2$ / 8 = 202 kipft (274 KNm)
T = M / d = (202 / 90 ft) = 2.24 kips (9.96 KN)

$w_{7\text{-}8}$ = 293 plf (4.27 KN/m)
M = $wL^2/8$ = $(0.293)(40\ ft)^2$ / 8 = 58.6 kipft (79.5 KNm)
T = M / d = (58.6 / 10 ft) = 5.86 kips (26.1 KN)

Seismic forces to vertical resisting elements from lower sloped roof diaphragm

Transverse direction: The horizontally braced diaphragms are assumed to act as rigid diaphragms spanning between the steel rigid frames and the end shear walls along lines 2 and 7. Due to the low stiffness of the moment frames as compared to that of the end concrete shear walls, it is assumed that all of the shear is distributed to the end walls in relation to their relative rigidities.

Determine relative rigidities of concrete shear walls at ends of low sloped roof area:

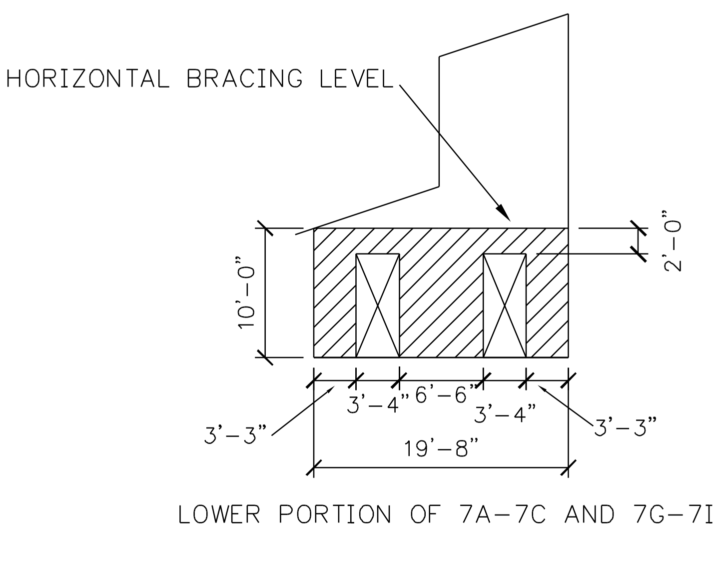
Wall Rigidity Equations

$f'_c := 3000\,psi$	Concrete strength = 3000 psi
$E_c := 57000\sqrt{\dfrac{f'_c}{psi}}\cdot psi$	ACI 318 Section 8.5.1
$E_c = 3122\,ksi$ $\quad E_c = 21.5251\,\dfrac{KN}{mm^2}$	Elastic Modulus of concrete
$E_v := 0.4\cdot E_c$ $\quad E_v = 1249\,ksi$	Shearing Modulus of concrete
$t := 8\cdot in$ $\quad E_v = 8.61\,\dfrac{KN}{mm^2}$	Wall thickness
$P := 1\cdot kip$	Shear load to determine pier stiffness

Concrete Functions:

$A(d) := t\cdot d$	Area of wall pier
$I(d) := \dfrac{1}{12}\cdot t\cdot d^3\cdot 0.8$	Assume stiffness = 80% of I per FEMA 273 Sec. 6.8.2.2
$\Delta_c(h,d) := \dfrac{(1.2\cdot P\cdot h)}{A(d)\cdot E_v} + \dfrac{P\cdot h^3}{3\cdot E_c\cdot I(d)}$	Deflection of a cantilevered pier
$\Delta_f(h,d) := \dfrac{(1.2\cdot P\cdot h)}{A(d)\cdot E_v} + \dfrac{P\cdot h^3}{12\cdot E_c\cdot I(d)}$	Deflection of a fixed-fixed pier
$R_f(h,d) := \dfrac{1}{\Delta_f(h,d)}$	Rigidity of fixed wall pier

Shear Walls 7A-7C and 7G-7I:



LOWER PORTION OF 7A-7C AND 7G-7I

1 foot = 0.305m

Solid wall ABCD $\Delta_{solid} := \Delta_c(10 \cdot ft, 19.67 \cdot ft)$ $\Delta_{solid} = 8.737 \cdot 10^{-5} \cdot in$

Subtract strip BCD $\Delta_{bcd} := \Delta_f(8 \cdot ft, 19.67 \cdot ft)$ $\Delta_{bcd} = 5.2219 \cdot 10^{-5} \cdot in$

$\Delta_a := \Delta_{solid} - \Delta_{bcd}$ $\Delta_a = 3.5151 \cdot 10^{-5} \cdot in$

Add back in piers B, C and D $R_b := R_f(8 \cdot ft, 3.25 \cdot ft)$ $R_b = 959.581 \cdot \frac{1}{in}$

$R_d := R_b$ $R_d = 959.581 \cdot \frac{1}{in}$

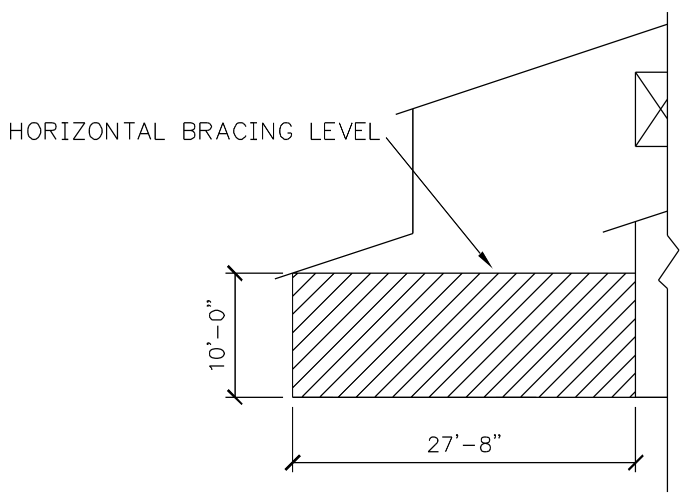
$R_c := R_f(8 \cdot ft, 6.5 \cdot ft)$ $R_c = 4146.9681 \cdot \frac{1}{in}$

$\Delta_{bcd} := \frac{1}{R_b + R_c + R_d}$ $\Delta_{bcd} = 0.0002 \cdot in$

$\Delta_{total} := \Delta_a + \Delta_{bcd}$ $\Delta_{total} = 0.0002 \cdot in$

Rigidity of wall $R_{wall} := \frac{1 \cdot kip}{\Delta_{total}}$ $R_{wall} = 4999.9934 \cdot \frac{kip}{in}$ $R_{wall} = 875.6126 \cdot \frac{KN}{mm}$

Shear walls 2A-2D & 2F-2I



SHEAR WALLS 2A-2D AND 2F-2I

Solid wall $\Delta_{solid} := \Delta_c(10 \cdot ft, 27.67 \cdot ft)$ $\Delta_{solid} = 5.28593 \cdot 10^{-5} \cdot in$

Rigidity of wall $R_{wall} := \frac{1 \cdot kip}{\Delta_{solid}}$ $R_{wall} = 18918.15 \cdot \frac{kip}{in}$ $R_{wall} = 3313 \cdot \frac{KN}{mm}$

Distribute the shear force tributary to the lower sloped roof areas to the shear walls.

Total weight tributary to the lower sloped roof areas = 216.7 kips (964KN)
Weight of each lower sloped roof area = 216.7 / 2 = 108.4 kips (482 KN)

Shear force tributary to each lower sloped roof area = C_sW = 0.167(108.4 kips) = 18.1 kips (80.5 KN)

$R_{2A\text{-}2D}$ = 18918 kips / in $R_{7A\text{-}7C}$ = 5000 kips / in $V_{element} = V \dfrac{R_{element}}{\sum R}$

$V_{2A\text{-}2D}$ = 18.1(18918) / (18918+5000) = 14.3 kips (63.6KN)
$V_{7A\text{-}7C}$ = 18.1(5000)/(18918+5000) = 3.78 kips (16.8 KN)

Due to symmetry, $V_{2F\text{-}2I} = V_{2A\text{-}2D}$ = 14.3 kips (63.6KN) $V_{7G\text{-}7I} = V_{7A\text{-}7C}$ = 3.78 kips (16.8 KN)

The horizontal bracing is assumed to act as a rigid support to the transverse moment frames. It is further assumed that the end shear walls (along grid lines 2 and 7) are infinitely rigid (relative to the frames). Therefore, the horizontally braced diaphragms of the low sloped roof will transmit the shear forces from the moment frames (shear forces tributary to the high roof) into the end shear walls.

Shear in moment frames from upper roof diaphragm = 4 (4.81 k) = 19.24 k (41.4 KN)

There are two low sloped roof areas (2A-B7 & H2-I7); assume each resists ½ of this shear force.

Shear to each low sloped roof area = ½(19.24k) = 9.62 k (42.8KN)

Distribute shear to walls based on rigidity;

$V_{2A\text{-}2D}$ = 9.62(18918) / (18918+5000) = 7.61 kips (33.8 KN)
$V_{7A\text{-}7C}$ = 9.62(5000)/(18918+5000) = 2.01 kips (8.94 KN)

Due to symmetry, $V_{2F\text{-}2I} = V_{2A\text{-}2D}$ = 7.61 kips (33.8 KN) $V_{7G\text{-}7I} = V_{7A\text{-}7C}$ = 2.01 kips (8.94 KN)

Total Shear to walls from lower roof diaphragms;
$V_{2A\text{-}2D}$ = 14.3 k + 7.61 k = 21.91 k (97.5 KN) $V_{7A\text{-}7C}$ = 3.78 k + 2.01 k = 5.79 k (25.8 KN)
Due to symmetry, $V_{2F\text{-}2I} = V_{2A\text{-}2D}$ = 21.91 kips (95.7 KN) $V_{7G\text{-}7I} = V_{7A\text{-}7C}$ = 5.79 kips (25.8 KN)

Longitudinal direction: The shear walls along grid lines A and I resist all of the shear tributary to the low sloped roofs.

Diaphragm force:

Weight tributary to each low sloped roofs (minus weight of exterior shear wall A or I):

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.^{2})	Unit Weight (psf / lb)	Seismic Weight (kips)
LOWER ROOF						
Roof A2-B7	10.0	90.0	100.0	900.0	16.9	15.2
TRANSVERSE WALLS	0.0	0.0	0.0	0.0	0.0	0.0
Wall 7A-7B	---	---	---	62.0	100.0	6.2
Wall 2A-2B	---	---	---	62.0	100.0	6.2
LONGITUDINAL WALLS	0.0	0.0	0.0	0.0	0.0	0.0
Wall B2-B7	6.5	90.0	65.0	380.3	100.0	38.0
					TOTAL	65.61
						292 KN

V = C_sW = 0.167(65.61 kips) = 10.96 k (48.8 KN)

This shear force must be transferred to the shear walls (A2-A7 & I2-I7) through the horizontal bracing. The bracing also transfers the shear force from wall lines B & H (tributary to the upper roof diaphragm).

Total shear force transferred through horizontal bracing = 19.8 k + 10.96 kips = 30.8 kips (137 KN)

Total shear to walls A2-A7 and I2-I7 = Horizontal bracing shear + shear due to self-weight:

Weight of wall A2-A7 = 42.75 kips
Shear due to self-weight = C_sW = (0.167)(42.75k) = 7.14 k (318 KN)

Total shear to walls A2-A7 & I2-I7 trib. to high and low sloped roofs = 30.8 k + 7.14 k = 37.9 k (169 KN)

Chord forces:
The low sloped roof diaphragm areas are conservatively assumed to span 90 feet (27.45 m) between shear wall lines 2 and 7 for transverse seismic forces. An equivalent running load, w, is found by dividing the total shear to the diaphragm by the span. The shear is made up of forces tributary to the low sloped roof areas and the reactions at the moment frames.

Shear force tributary to the low sloped roof areas = 18.1 kips (80.5KN)
Shear force to each diaphragm from moment frame reactions = 4(4.81 k) / 2 = 9.62 kips (42.8KN)
Total shear force = 18.1 k + 9.62 k = 27.7 kips (123 KN)
Equivalent running load, w = V / L = 27.7 / 90’ = 308 plf (4.49 KN/m)
Moment = $wL^2 / 8 = (.308)(90)^2 / 8$ = 312 kft (423 KNm)
T = M / d = 312 / 9.67’ = 32.3 kips (143 KN)

Chord forces for longitudinal seismic forces are negligible by inspection.

Collector forces:
The beams along grid lines B & H must collect the shear forces from the upper concrete shear / window walls and distribute them to the horizontal bracing of the low sloped roof area. The beams collect the shear force from the upper roof area and the shear force associated with wall lines B & H tributary to the low sloped roof areas.

Shear force from wall line B tributary to the upper roof diaphragm = 19.8 kips (88.1 KN)
Weight of wall line B tributary to the low sloped roof area = 15.2 k
Shear force from wall line B tributary to the low sloped roof area = C_sW = (0.167)(15.2k) = 2.54k (11.3KN)
Total shear force collected by beams along grid lines B & H = 19.8 + 2.54 = 22.34 kips (99.4 KN)
Unit collector shear force, v = V / L = 22.34 kips / 90’ = 248 plf (3.62 KN/m)
Collector force = $v*L_{collector}$ = (248 plf)(18’) = 4.46 kips (19.8 KN)

<u>Seismic forces to vertical resisting elements from entrance area diaphragm</u>

The concrete shear walls resist seismic shear forces tributary to the entrance area according to their tributary areas.

Transverse direction:

LOWER SLOPED ROOF @ ENTRANCE TRIBUTARY SEISMIC WEIGHTS (ROOF & NORMAL WALLS)

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)
LOWER ROOF						
Roof D1-F2	10.0	24.0	100.0	240.0	13.7	3.3
LONGITUDINAL WALLS						
Wall D1-D2	8.0	10.0	100.0	80.0	100.0	8.0
Wall F1-F2	8.0	10.0	100.0	80.0	100.0	8.0
					TOTAL	19.3

Total weight = 19.3 kips Seismic coefficient, C = 0.167 85.8 KN

Seismic force = C_sW = (0.167)(19.3) = 3.22 kips (14.3 KN)

Equivalent Running Load w = 3.22 k / 10 ft. = 322 plf (4.70 KN/m)

– Shear force to shear wall line 1 from entrance roof
  V = wL/2 = (322)(5’) = 1.61 kips (7.16 KN)

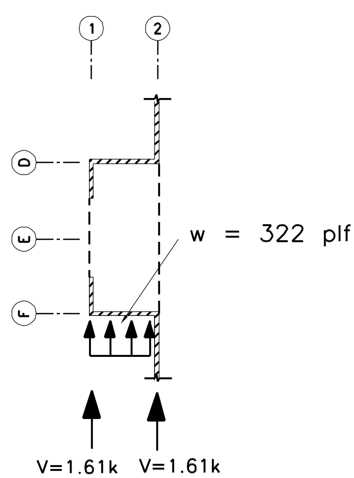
– Shear force to wall line 2 (same as wall line 1 due to symmetry)
  V = 1.61 kips (assume ½ goes to 2A-2D and ½ to 2F-2I = 0.805 (3.58 KN))

The shear walls along grid line 1 must also resist the forces associated with their self-weight. Wall line 2 has no seismic weight tributary to the entrance roof.

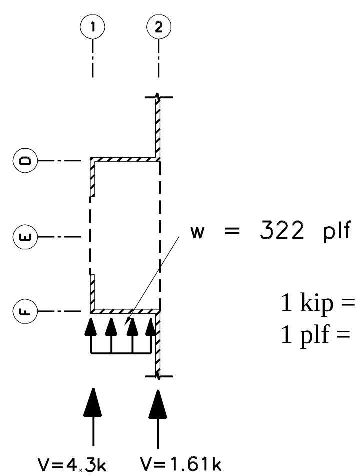
Weight of wall line 1 tributary to the entrance roof = 16.0 kips

Self-weight inertial force = C_sW = (0.167)(16.0) = 2.67 kips (11.88 KN)

Total shear to wall line 1 tributary to entrance roof diaphragm = (1.61 k) + (2.67 k) = 4.3 k (19.1 KN)



SHEAR TO VERTICAL ELEMENTS FROM ENTRANCE ROOF



1 kip = 4.448 KN
1 plf = 14.59 N/m

SHEAR TO VERTICAL ELEMENTS FROM ENTRANCE ROOF + SELF INERTIA FORCES

Longitudinal direction: Seismic forces in the longitudinal direction are resisted by shear wall lines D & F evenly due to symmetry. The diaphragm forces are due to the weight of the roof area and normal wall line 1. The weight of wall line 2 is included in the weight tributary to the upper roof.

LOWER SLOPED ROOF @ ENTRANCE TRIBUTARY SEISMIC WEIGHTS (ROOF AND NORMAL WALLS)

Item	Tributary Height / Width (ft.)	Length / Width (ft.)	% Solid	Area (ft.2)	Unit Weight (psf / lb)	Seismic Weight (kips)
LOWER ROOF						
Roof D1-F2	10.0	24.0	100.0	240.0	13.7	3.3
TRANSVERSE WALLS						
Wall D1-F1	---	---	100.0	160.0	100.0	16.0
					TOTAL	19.3
						85.6 KN

Total weight = 19.3 kips   Seismic coefficient, C = 0.167
Seismic force = C_sW = (0.167)(19.3) = 3.22 kips (14.3 KN)
Equivalent Running Load w = 3.22 k / 24 ft. = 134 plf (1.96 KN/m)

– Shear force to wall line D from entrance roof
  V = wL/2 = (134)(12') = 1.61 k (7.16 KN)

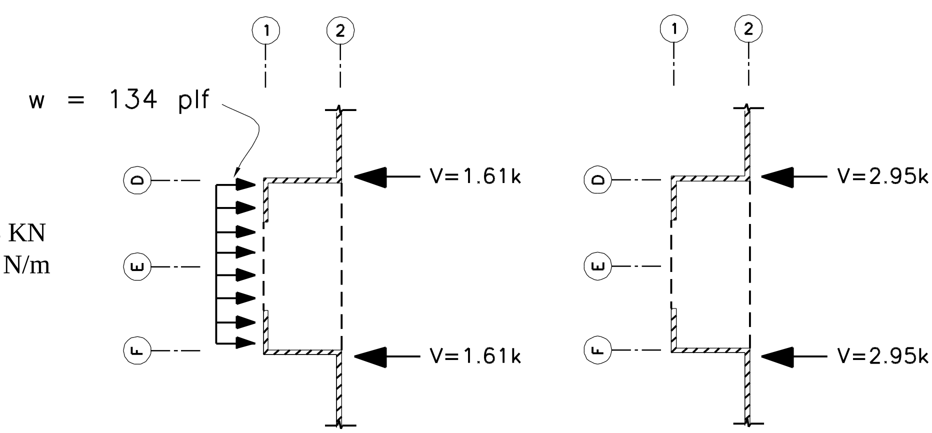
– Shear force to wall line F (same as wall line D)
  V = 1.61 k (7.16 KN)

The self-weight inertial effects of the shear walls due to the weight tributary to the entrance roof must now be added to the shears determined for shear wall lines D and F.

– Weight of wall line D tributary to the entrance roof = 8.0 kips
  Self-weight inertial force = C_sW = (0.167)(8.0) = 1.34 kips
  Total shear to wall line D tributary to entrance roof diaphragm = (1.61k) + (1.34k) = 2.95 k (13.1 KN)

– Wall line F same as line D
  Total shear = 2.95 k (13.1 KN)

1 kip = 4.448 KN
1 plf = 14.59 N/m



SHEAR TO VERTICAL ELEMENTS FROM ENTRANCE ROOF

SHEAR TO VERTICAL ELEMENTS FROM ENTRANCE ROOF + SELF INERTIA FORCES

Seismic forces to vertical resisting elements from sacristy area diaphragm

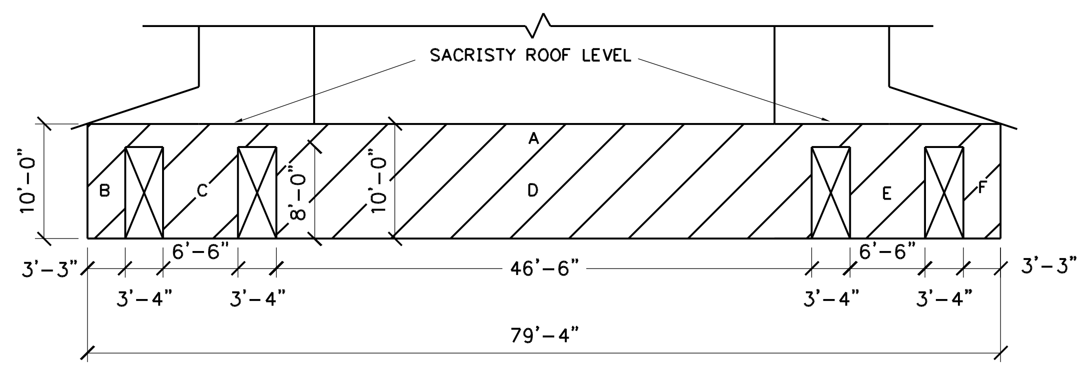
The concrete slab roofs of the sacristy areas act as rigid diaphragms. The shear force from these diaphragms are resisted by the vertical elements based on relative rigidities.

Transverse direction:

Shear wall line 7:

The rigidity of wall segments 7A-7C and 7G-7I was determined to be 5000 kips / in. (876 KN/mm) (see calculation in low sloped roof shear force section).

Shear wall line 8:



SHEAR WALL LINE 8 RIGIDITY (AT SACRISTY LEVEL)

Solid wall ABCDEF	$\Delta_{solid} := \Delta_c(10\cdot ft, 76.33ft)$	$\Delta_{solid} = 1.6186\cdot 10^{-5}\cdot in$
Subtract strip BCDEF	$\Delta_{bcdef} := \Delta_f(8\cdot ft, 76.33ft)$	$\Delta_{bcdef} = 1.2647\cdot 10^{-5}\cdot in$
	$\Delta_a := \Delta_{solid} - \Delta_{bcdef}$	$\Delta_a = 3.5398\cdot 10^{-6}\cdot in$
Add back in piers B, C, D, E and F	$R_b := R_f(8\cdot ft, 3.25ft)$	$R_b = 959.581\cdot\frac{1}{in}$
	$R_F := R_b$	$R_F = 959.581\cdot\frac{1}{in}$
	$R_c := R_f(8\cdot ft, 3.33ft)$	$R_c = 1017.8097\cdot\frac{1}{in}$
	$R_e := R_c$	$R_e = 1017.8097\cdot\frac{1}{in}$
	$R_d := R_f(8\cdot ft, 46.5ft)$	$R_d = 47801.829\cdot\frac{1}{in}$
	$\Delta_{bcdef} := \frac{1}{R_b + R_c + R_d + R_e + R_F}$	$\Delta_{bcdef} = 1.9321\cdot 10^{-5}\cdot in$
	$\Delta_{wall} := \Delta_a + \Delta_{bcdef}$	$\Delta_{wall} = 2.2861\cdot 10^{-5}\cdot in$

Rigidity of wall $\quad R_{wall} := \frac{1\cdot kip}{\Delta_{wall}} \quad R_{wall} = 43742.6663\frac{kip}{in} \quad R_{wall} = 7660.336\frac{KN}{mm}$

Shear wall line 8 receives shear from both sacristy areas. Therefore, it is assumed that ½ of the rigidity of the entire wall (=1/2 * 43743 = 21872 k/in) will be used in determining the distribution of forces to vertical resisting elements tributary to each sacristy area.

For each sacristy; $R_{wall\ 7}$ = 5000 kips / in (876 KN/mm) $R_{wall\ 8}$ = 21872 kips /in (7660 KN/mm)

Shear force tributary to each sacristy = 5.3 kips (23.6 KN)

R_7 = 5000 kips / in R_8 = 21872 kips / in $V_{element} = V \dfrac{R_{element}}{\sum R}$

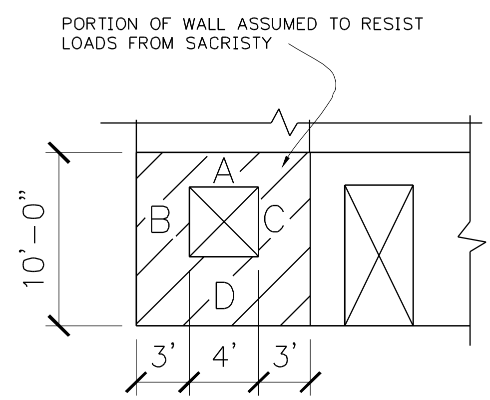
$V_{7A\text{-}7C}$ = 5.3(5000) / (21872+5000) = 0.99 kips (4.40 KN)
V_8= 5.3(21872)/(21872+5000) = 4.31 kips (19.2 KN)

Due to symmetry, $V_{7G\text{-}7I} = V_{7A\text{-}7C}$ = 0.99 kips (4.40 KN)

Total shear to wall line 8 (for forces trib. to sacristies) = 2(4.31 k) = 8.62 kips (38.3 KN)

Longitudinal Direction

Rigidity of shear wall lines A & I: It is assumed that only the end portion ot the walls (between grid lines 7 and 8) resist shear forces tributary to the sacristy areas (this is a very conservative assumption since the walls run the full length of the building).



1 foot = 0.305m

SHEAR WALL LINES A & I

Wall lines A & I at sacristy

Solid wall ABCD $\Delta_{solid} := \Delta_c(10\cdot ft, 10\cdot ft)$ $\Delta_{solid} = 0.0003 \cdot in$

Subtract strip BC $\Delta_{bc} := \Delta_f(4\cdot ft, 10\cdot ft)$ $\Delta_{bc} = 5.1249 \cdot 10^{-5} \cdot in$

$\Delta := \Delta_{solid} - \Delta_{bc}$ $\Delta = 0.0003 \cdot in$

Add back in piers B & C $R_b := R_f(4\cdot ft, 3\cdot ft)$ $R_b = 3587.0055 \cdot \frac{1}{in}$

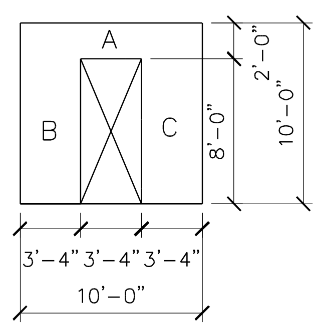
$R_c := R_f(4\cdot ft, 3\cdot ft)$ $R_c = 3587.0055 \cdot \frac{1}{in}$

$\Delta_{bc} := \frac{1}{R_b + R_c}$ $\Delta_{bc} = 0.0001 \cdot in$

$\Delta_{wall} := \Delta + \Delta_{bc}$ $\Delta_{wall} = 0.0004 \cdot in$

$R_{wall} := \frac{1\cdot kip}{\Delta_{wall}}$ $R_{wall} = 2448 \cdot \frac{kip}{in}$ $R_{wall} = 429 \cdot \frac{KN}{mm}$

Rigidity of lower portions of walls C7-C8 and G7-G8



LOWER PORTIONS OF WALLS
C7-C8 AND G7-G8

1 foot = 0.305m

Solid wall ABCD $\quad \Delta_{solid} := \Delta_c(10 \cdot ft, 10 \cdot ft) \quad \Delta_{solid} = 0.0003 \cdot in$

Subtract strip BCD $\quad \Delta_{bcd} := \Delta_f(8 \cdot ft, 10 \cdot ft) \quad \Delta_{bcd} = 0.0001 \cdot in$

$\Delta_a := \Delta_{solid} - \Delta_{bcd} \quad \Delta_a = 0.0002 \cdot in$

Add back in piers B and C $\quad R_b := R_f(8 \cdot ft, 3.33 \cdot ft) \quad R_b = 1017.8097 \cdot \frac{1}{in}$

$R_c := R_b \quad R_c = 1017.8097 \cdot \frac{1}{in}$

$\Delta_{bc} := \frac{1}{R_b + R_c} \quad \Delta_{bc} = 0.0005 \cdot in$

$\Delta_{total} := \Delta_a + \Delta_{bc} \quad \Delta_{total} = 0.0007 \cdot in$

Rigidity of wall $\quad R_{wall} := \frac{1 \cdot kip}{\Delta_{total}} \quad R_{wall} = 1449.6 \cdot \frac{kip}{in} \quad R_{wall} = 253.9 \cdot \frac{KN}{mm}$

For each sacristy;

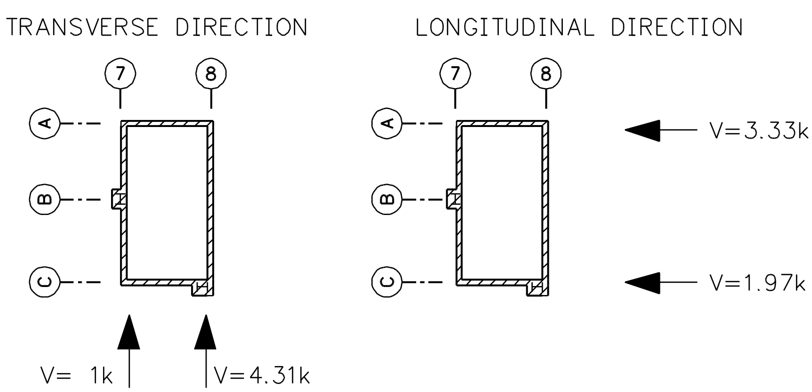
Shear force tributary to each sacristy = 5.3 kips (23.6 KN)

$R_{A7\text{-}A8}$ = 2448 kips / in $\quad R_{C7\text{-}C8}$ = 1450 kips /in $\quad V_{element} = V \frac{R_{element}}{\sum R}$

$V_{A7\text{-}A8}$ = 5.3(2448) / (2448 + 1450) = 3.33 kips (14.8 KN)
$V_{C7\text{-}C8}$= 5.3(1450)/(2448+1450) = 1.97 kips (8.76 KN)

Due to symmetry, $V_{I7\text{-}I8} = V_{A7\text{-}A8}$ = 3.33 kips (14.8 KN) $\quad V_{G7\text{-}G8} = V_{C7\text{-}C8}$ = 1.97 kips (8.97 KN)

1 kip = 4.448 KN



B-6 *Determine cr and cm*

The sacristy and lower sloped roof areas have rigid diaphragms. The torsional forces to the vertical resisting elements are calculated by finding the tributary mass and stiffness eccentricities.

<u>Sacristy</u>

Center of Mass

Element	Weight (kips)	x (ft.)	y (ft.)	Wx (kip*ft)	Wy (kip*ft)
Roof Deck	10.4	5.0	9.8	52.1	102.5
Wall 8A-8C	8.0	10.0	9.8	80.0	78.4
Wall A7-A8	3.8	5.0	19.7	18.8	73.8
Wall C7-C8	9.8	5.0	0.0	48.8	0.0
S =	31.9			199.6	254.6

$$cm = \sum Wx / \sum w$$

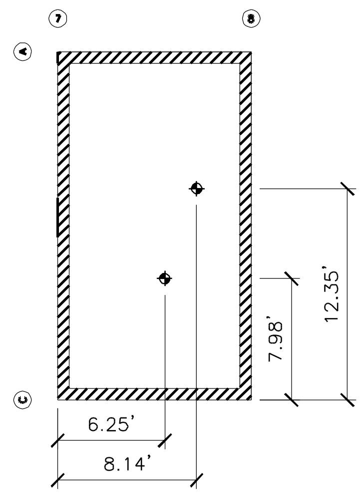
cm_x = (199.6) / (31.9) = 6.25’ (1.91m)
cm_y = (254.6) / (31.9) = 7.98’ (2.43m)

Center of Rigidity

Element	R_{cx} (kip / in)	R_{cy} (kip / in)	x (ft.)	y (ft.)	yR_{cx}	xR_{cy}
Wall A7-A8	2448	0	0	19.67	48152.2	0
Wall C7-C8	1450	0	0	0	0.0	0
Wall 7A-7C	0	5000	0	0	0.0	0
Wall 8A-8C	0	21872	10	0	0.0	218720
S =	3898	26872			48152	218720

$$cr = \sum xR / \sum R$$

cr_x = (48152) / (3898) = 12.35’ (3.77m)
cr_y = (218720) / (26872) = 8.14’ (2.48m)



1 foot = 0.305m

CENTER OR MASS & RIGIDITY FOR SACRISTY AREAS

<u>Lower sloped roof areas</u>

Center of mass:

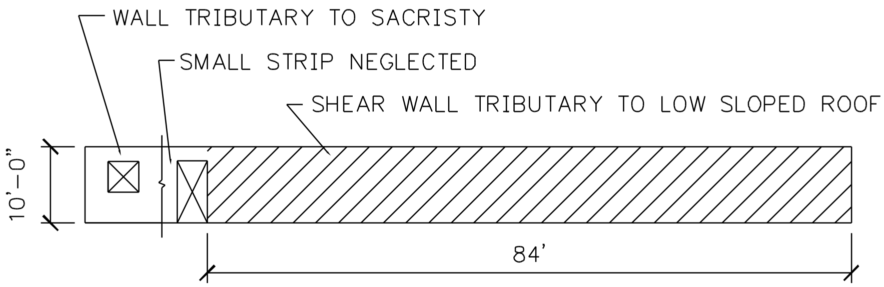
Element	Weight (kips)	x (ft.)	y (ft.)	Wx (kip*ft)	Wy (kip*ft)
Roof Deck	15.2	45.0	4.8	683.1	73.4
Wall A2-A7	42.8	45.0	9.7	1923.8	413.4
Wall B2-B7	38.0	45.0	0.0	1711.1	0.0
Wall 2A-2B	6.2	0.0	4.8	0.0	30.0
Wall 7A-7B	6.2	90.0	4.8	558.0	30.0
S =	108.4			4876.0	546.7

$cm = \sum Wx / \sum w$

cm_x = (4876) / (108.4) = 45’ (13.73m)
cm_y = (546.7) / (108.4) = 5.04’ (1.54m)

Center of rigidity

The rigidity of shear wall lines A and I must be calculated;



SHEAR WALL LINES A & I

Solid wall $\Delta_{solid} := \Delta_c(10\cdot ft, 84\cdot ft)$ $\Delta_{solid} = 1.4637 \times 10^{-5} \cdot in$

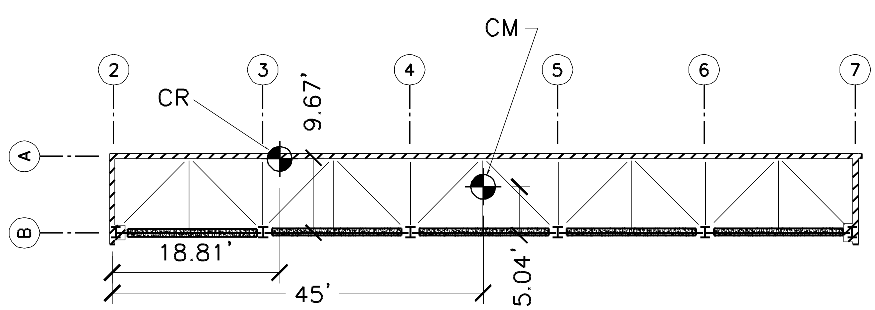
Rigidity of wall $R_{wall} := \dfrac{1\cdot kip}{\Delta_{solid}}$ $R_{wall} = 68320 \cdot \dfrac{kip}{in}$ $R_{wall} = 11964 \cdot \dfrac{KN}{mm}$

Element	R_{cx} (kip / in)	R_{cy} (kip / in)	x (ft.)	y (ft.)	yR_{cx}	xR_{cy}
Wall A2-A7	68320	0	0	9.67	660654.4	0
Wall 2A-2D	0	18918	0	0	0	0
Wall 7A-7C	0	5000	90	0	0	450000
S =	68320	23918			660654	450000

$cr = \sum xR / \sum R$

cr_x = (660654) / (68320) = 9.67' (2.95 m)
cr_y = (450000) / (23918) = 18.81' (5.74 m)

1 foot = 0.305 m



CENTER OF MASS AND RIGIDITY FOR LOW SLOPED ROOF AREAS

B-7 *Perform torsional analysis*

Sacristy Areas

Due to symmetry both of the sacristies will behave the same in torsion. Only one of the sacristies is analyzed and the results are used for both. The torsional eccentricity is taken as the offset between the centers of mass and rigidity and an additional 5% accidental eccentricity.

Torsional force resisted by each vertical element = $F_T = T\frac{Rd}{\sum Rd^2}$

Transverse Forces:

Center of Mass in X direction = 6.25 ft.
Center of Rigidity in X direction = 8.14 ft.
Actual Eccentricity in X direction = 1.89 ft.
Perpendicular Span in X direction = 10 ft.
Accidental Eccentricity (5% of Span) = 0.5 ft.
Design Eccentricity in X direction = 2.39 ft. (0.73m)
Seismic Force Tributary to each Sacristy = 5.3 kips (23.6 KN)
Torsional Force (V * e) = 12.70 kip*ft (17.2 KNm)

Element	R	d	Rd	Rd^2	Torsional Force (kip)
Wall 7A-7C	5000	8.14	40697	331243	0.68
Wall 8	21872	-1.86	-40697	75723	-0.68*
Wall A7-A8	2448	7.32	17912	131061	0.30
Wall C7-C8	1450	12.35	17912	221267	0.30
			Σ =	759294	

*Note: Negative torsional force contributions are neglected.

Longitudinal Forces:

Center of Mass in Y direction = 7.98 ft.
Center of Rigidity in Y direction = 12.35 ft.
Actual Eccentricity in Y direction = 4.38 ft.
Perpendicular Span in Y direction = 19.67 ft.
Accidental Eccentricity (5% of Span) = 0.9835 ft.
Design Eccentricity in Y direction = 5.36 ft. (1.63m)
Seismic Force Tributary to each Sacristy = 5.3 kips (23.6 KN)
Torsional Force (V * e) = 28.52 kip*ft (38.7 KNm)

Element	R	d	Rd	Rd^2	Torsional Force (kip)
Wall 7A-7C	5000	8.14	40697	331243	1.529
Wall 8	21872	1.86	40697	75723	1.529
Wall A7-A8	2448	-7.32	-17912	131061	-0.673*
Wall C7-C8	1450	12.35	17912	221267	0.673
			Σ =	759294	

*Note: Negative torsional force contributions are neglected.

Lower Sloped Roof Areas

Due to symmetry both of the lower sloped roof areas will behave the same in torsion. Only one of the areas is analyzed and the results are used for both. In addition to the torsion created between the offset between the centers of mass and rigidity, the transfer of shear into the diaphragm from vertical resisting elements tributary to the upper roof diaphragm creates torsion. In the transverse direction the braced moment frames transfer their shear into the horizontal diaphragm. This creates a torsional force equal to the shear transferred times the distance between the center of application of the moment frame forces and the center of rigidity (45'-18.81'= 26.2'). In the longitudinal direction the offset between the upper shear walls (along lines B & H) and the lower resisting elements (walls along lines A & I) creates torsional forces.

Transverse Forces:

Center of Mass in X direction =	45.00 ft.	
Center of Rigidity in X direction =	18.81 ft.	
Actual Eccentricity in X direction =	26.19 ft.	
Perpendicular Span in X direction =	90 ft.	
Accidental Eccentricity (5% of Span) =	4.5 ft.	
Design Eccentricity in X direction =	30.69 ft.	
Seismic Force Tributary to each Diaphragm =	18.1 kips	
Eccentricity Torsional Force (V * e) =	554.2 kipft	
Shear Force from Upper Roof (moment frames) =	19.24 kips	=4 x 4.81
Distance to Center of Rigidity =	26.19 ft	
Torsion Force from Upper Roof =	503.8 kipft	
Total Torsion =	1058.0 kipft (1434 KNm)	

Element	R	d	Rd	Rd^2	Torsional Force (kip)
Wall A2-A7	68320	0.00	0	0	0.0
Wall 2A-2D	18918	-18.81	-355848	6693493	-11.8*
Wall 7A-7C	5000	71.19	355950	25340081	11.8
			Σ =	32033573	

*Note: Negative torsional force contributions are neglected.

Longitudinal Forces:

Center of Mass in Y direction =	5.05 ft.
Center of Rigidity in Y direction =	9.67 ft.
Actual Eccentricity in Y direction =	4.62 ft.
Perpendicular Span in Y direction =	9.67 ft.
Accidental Eccentricity (5% of Span) =	0.4835 ft.
Design Eccentricity in Y direction =	5.11 ft.
Seismic Force Tributary to each Diaphragm =	18.1 kips
Eccentricity Torsional Force (V * e) =	92.25 kipft
Shear Force from Upper Roof (wall line B) =	19.8 kips
Distance to Center of Rigidity =	9.67 ft
Torsion Force from Upper Roof =	191.5 kipft
Total Torsion =	283.7 kipft (385 KNm)

Element	R	d	Rd	Rd^2	Torsional Force (kip)
Wall A2-A7	68320	0.00	0	0	0.0
Wall 2A-2D	18918	18.81	355848	6693493	3.2
Wall 7A-7C	5000	71.19	355950	25340081	3.2
			Σ =	32033573	

Total Shear Force to Shear Walls:

1 kip = 4.448 KN

Transverse Forces

Element	Shear from Upper Roof (kips)	Shear from Lower Roof (kips)	Shear from Entrance Roof (kips)	Shear from Sacristy (kips)	Torsional Shear from Lower Roof (kips)	Torsional Shear from Sacristy (kips)	Total Shear force (kips)
Wall 1D-1F	0	0	4.3	0	0	0	4.3
Wall 2A-2D	7.9	21.91	0.805	0	0	0	30.6
Wall 2F-2I	7.9	21.91	0.805	0	0	0	30.6
Wall 7A-7C	3.015	5.79	0	0.99	11.8	0.68	22.3
Wall 7G-7I	3.015	5.79	0	0.99	11.8	0.68	22.3
Wall 8A-8I	12.3	0	0	8.62	0	0	20.9
Wall D1-D2	0	0	0	0	0	0	0.0
Wall F1-F2	0	0	0	0	0	0	0.0
Wall A2-A7	0	0	0	0	0	0	0.0
Wall I2-I7	0	0	0	0	0	0	0.0
Wall B2-B7	0	0	0	0	0	0	0.0
Wall H2-H7	0	0	0	0	0	0	0.0
Wall A7-A8	0	0	0	0	0	0.3	0.3
Wall I7-I8	0	0	0	0	0	0.3	0.3
Wall C7-C8	0	0	0	0	0	0.3	0.3
Wall G7-G8	0	0	0	0	0	0.3	0.3

Longitudinal Forces

Element	Shear from Upper Roof (kips)	Shear from Lower Roof (kips)	Shear from Entrance Roof (kips)	Shear from Sacristy (kips)	Torsional Shear from Lower Roof (kips)	Torsional Shear from Sacristy (kips)	Total Shear force (kips)
Wall 1D-1F	0	0	0	0	0	0	0
Wall 2A-2D	0	0	0	0	3.2	0	3.2
Wall 2F-2I	0	0	0	0	3.2	0	3.2
Wall 7A-7C	0	0	0	0	3.2	1.53	4.73
Wall 7G-7I	0	0	0	0	3.2	1.53	4.73
Wall 8A-8I	0	0	0	0	0	3.06	3.06
Wall D1-D2	0	0	2.95	0	0	0	2.95

Wall F1-F2	0	0	2.95	0	0	0	2.95
Wall A2-A7	19.8	18.1	0	0	0	0	37.9
Wall I2-I7	19.8	18.1	0	0	0	0	37.9
Wall B2-B7	19.8	0	0	0	0	0	19.8
Wall H2-H7	19.8	0	0	0	0	0	19.8
Wall A7-A8	0	0	0	3.33	0	0	3.33
Wall I7-I8	0	0	0	3.33	0	0	3.33
Wall C7-C8	6.86	0	0	1.97	0	0.67	9.5
Wall G7-G8	6.86	0	0	1.97	0	0.67	9.5

B-8 *Determine need for redundancy factor, r.*

Transverse Direction: Seismic forces in the transverse direction are resisted by a combination of concrete shear walls and steel moment frames. The majority of the shear force is resisted by shear wall line 8. For shear walls, r_{max} is equal to the shear in the most heavily loaded wall multiplied by $10/l_w$, divided by the story shear:

$$\rho = 2 - \frac{20}{r_{max}\sqrt{A}} \quad , \; r_{max} = \frac{V}{V_T}\frac{10}{l_w}$$

$$r_{max} = \frac{20.9}{106}\frac{10}{40'} = 0.05\,, \quad \rho = 2 - \frac{20}{0.05\sqrt{8174}} = -2.4\,, \text{ use } 1.0$$

Longitudinal Direction Seismic forces in the longitudinal direction are resisted entirely by concrete shear walls.

$$r_{max} = \frac{57.7}{106}\frac{10}{84} = 0.065\,, \quad \rho = 2 - \frac{20}{0.065\sqrt{8174}} = -1.40\,, \text{ use } 1.0$$

B-9 *Determine need for overstrength factor, W_o*

The overstrength factor is used for the design of the collectors (beams along lines B and H that support the upper window/shear walls.

B-10 *Calculate combined load effects*

The load combinations from ASCE 7-95 are:

(1) 1.4D
(2) 1.2D + 1.6L + 0.5L_r
(3) 1.2D + 0.5L + 1.6L_r
(4) 1.2D + E + 0.5L
(5) 0.9D + E

Where $E = \rho Q_E \pm 0.2\, S_{DS}D$ Eq. 4-4 & 4-5

When specifically required by FEMA 302 (Collectors, their connections, and bracing connections for this example) the design seismic force is defined by:

$E = \Omega_0 Q_E \pm 0.2 S_{DS}D$ Eq. 4-6 & 4-7

The term $0.2S_{DS}D$ is added to account for the vertical earthquake accelerations.

$0.2S_{DS}D = 0.2(1.0)D = 0.2\ D$
Therefore, 0.2 will be added to the dead load factor for load combinations 4 and 5.

B-11 *Determine structural member sizes*

(a) *Upper roof diaphragm forces* - The upper roof diaphragm consists of metal decking. The decking is selected from a manufacture's catalog with the required shear and gravity capacities.

Transverse direction:
Diaphragm shear force demand = 4.21k (18.7 KN)
Diaphragm shear depth = 60' (18.3m)
Unit shear demand = (4.21k) / (60') = 70 plf (1021 N/m)

Longitudinal direction:
Diaphragm shear force demand (grid lines 2-7) = 14.5 k
Diaphragm shear depth (grid lines 2-7) = 90'
Unit shear demand (grid lines 2-7) = (14.5k) / (90') = 161 plf (2.35 KN/m)
Diaphragm shear force demand (grid lines 7-8) = 6.42 k
Diaphragm shear depth (grid lines 7-8) = 10'
Unit shear demand (grid lines 7-8) = (6.42k) / (10') = 642 plf (9.37 KN/m)

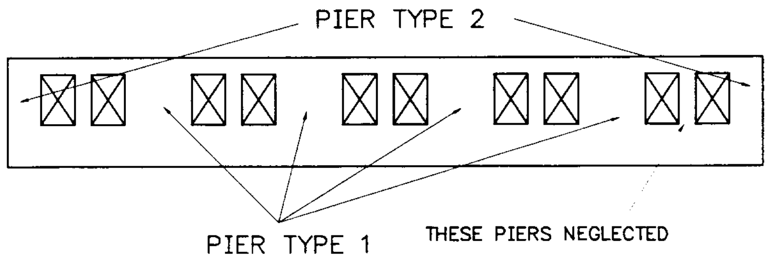
(b) *Reinforced Concrete Shear Walls*
Some of the shear walls resist forces in both the transverse and longitudinal direction due to torsion. Therefore, per FEMA 302 Section 5.2.7 orthogonal effects must be considered. The walls are checked for 100% of the force in one direction plus 30% in the orthogonal direction.

Element	Total Transverse Shear (kips)	Total Longitudinal Shear (kips)	100% Transverse + 30% Longitudinal (kips)	100% Longitudinal + 30% Transverse (kips)
Wall 1D-1F	4.3	0.0	4.3	1.3
Wall 2A-2D	30.6	3.2	31.6	12.4
Wall 2F-2I	30.6	3.2	31.6	12.4
Wall 7A-7C	22.3	4.7	23.7	11.4
Wall 7G-7I	22.3	4.7	23.7	11.4
Wall 8A-8I	20.9	3.1	21.8	9.3
Wall D1-D2	0.0	3.0	0.9	3.0
Wall F1-F2	0.0	3.0	0.9	3.0
Wall A2-A7	0.0	37.9	11.4	37.9
Wall I2-I7	0.0	37.9	11.4	37.9
Wall B2-B7	0.0	19.8	5.9	19.8
Wall H2-H7	0.0	19.8	5.9	19.8
Wall A7-A8	0.3	3.3	1.3	3.4
Wall I7-I8	0.3	3.3	1.3	3.4
Wall C7-C8	0.3	9.5	3.2	9.6
Wall G7-G8	0.3	9.5	3.2	9.6

1 kip 4.448 KN

Supported concrete shear / window walls - The supported concrete walls along grid lines B & H transfer the shear forces from the upper roof diaphragm to the beam collectors along the same grid lines. The shear from the collectors is then transferred to exterior shear wall lines A & I through horizontal bracing.

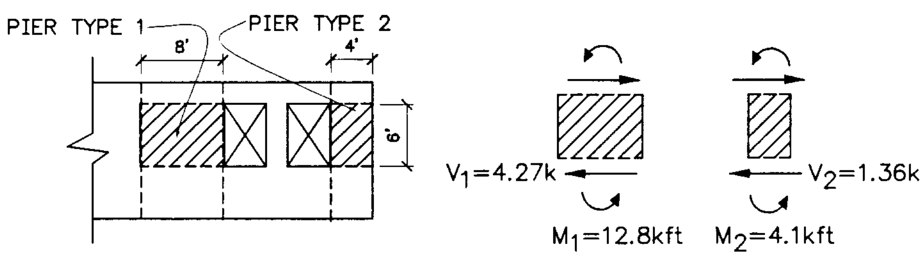
Shear from upper roof diaphragm = 19.8k (88.1 KN)



The pier elements resist shear in relation to their relative rigidities. It is assumed that the slender piers between the windows resist no shear.

R_1 = 8993 kips / in. (157 KN/mm) R_2 = 2865 kips / in. (502 KN/mm) (calcs. not shown)
ΣR = 4(8993) + 2(2865) = 41702 kips / in. (7303 KN/mm)

V_1 = 19.8(8993/41702) = 4.27 k (19.0 KN) V_2 = 19.8(2865/41702) = 1.36 k (6.05 KN)
M_1 = VL/2 = (4.27)(6) / 2 = 12.8 kft (17.4 KNm) M_2 = (1.36)(6) / 2 = 4.1 kft (5.56 KNm)



Shear strength of concrete: $V_n = A_{cv}\left(2\sqrt{f_c'} + \rho_n f_y\right)$ (ACI 318 Eq. 21-6)

ACI 318 Sec. 21.6.2.1 requires that $\rho > 0.0025$ for shear stress > $A_{cv}\sqrt{f_c'} = A_{cv}\sqrt{3000} = A_{cv}(54.8\text{psi})$
Assume all walls have stress exceeding this value (conservative). (378 KN/m^2)
Use 2-#5 bars at 12" in each direction for all walls (ρ=(0.31in.^{2}/(12 x 8) = 0.003 > 0.0025))
$V_n = A_{cv}\left(2\sqrt{3000} + 0.003(60000)\right) = A_{cv}(290\text{psi})$ or (2000 KN/m^2)

$\phi V_n = 0.6(290)A_{cv} = A_{cv}(174\text{ psi})$ (1200 KN/m^2) (ϕ = 0.6 per ACI 318 Sec. 9.3.4.1)
Shear strength of pier type 1 = (96")(8")(174) = 134 k (596 KN) > 4.27 k (19.0 KN), OK
Shear strength of pier type 2 = (48")(8")(174) = 67 k (298 KN) > 1.36 k (6.05 KN), OK

Check need for boundary zones; (per FEMA 302 Sec. 9.1.1.13)
Section 9.1.1.13 of FEMA 302 exempts walls & wall segments that meet the following conditions from needing boundary zones;

1. $P_u \le 0.10A_g f'_c$ (all wall segments in the structure meet this requirement since they are non bearing with low axial loads).

and either:

2. $M_u/V_u l_w \le 1.0$ or
$V_u \le 3A_{cv}\sqrt{f_c'} = A_{cv}(164\text{psi})$ and $M_u/V_u l_w \le 3.0$

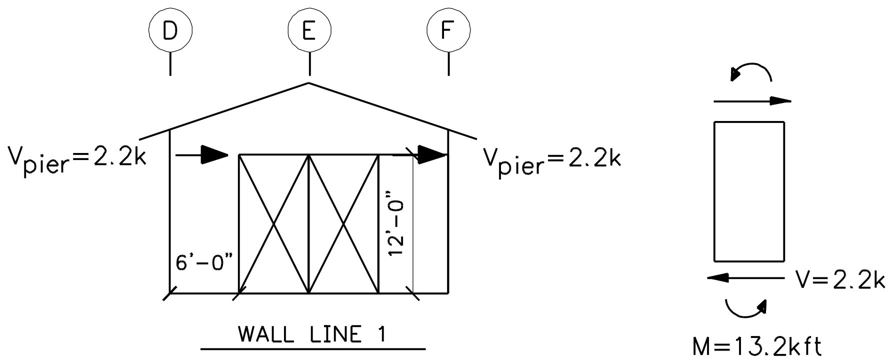
All of the shear walls have low shear stresses due to large amount of wall length; thus, by inspection,

the $V_u \leq 3A_{cv}\sqrt{f'_c} = A_{cv}(164\text{psi})$ term is satisfied. Therefore, no boundary zones will be required in any wall segment if $M_u/V_u l_w \leq 3.0$ is satisfied.
Pier 1 M/Vl = 12.8/4.27(8) = 0.37 <3.0 (No boundary zones required)
Pier 2 M/Vl = 4.1/1.36(4) = 0.75 <3.0 (No boundary zones required)

Shear wall line 1

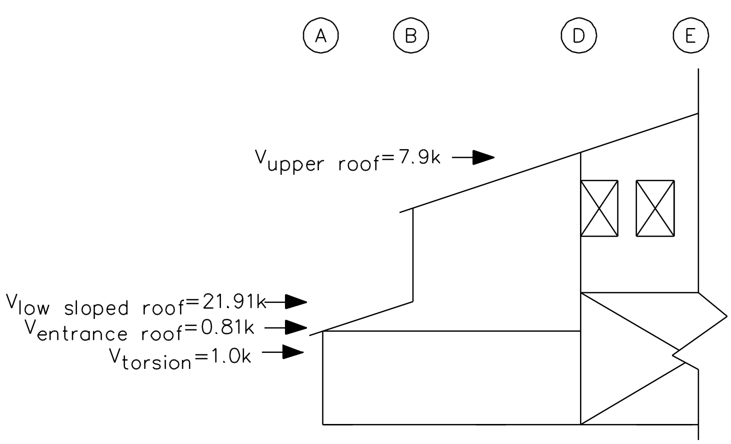
Shear wall line 1 resists forces tributary to the entrance only. The wall line consists of two pier elements which each resist ½ of the total loads.

$V_{wall\ 1}$ = 4.3 kips $V_{each\ pier}$ = ½(4.3) = 2.2 k (9.79 KN)
M = Vl / 2 = (2.2)(12') / 2 = 13.2kft (17.90 KNm)



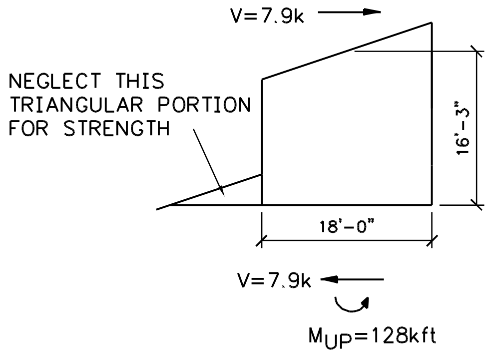
Shear strength of pier = (72")(8")(174) = 100 k (445 KN) > 2.2 k (9.79 KN), OK
Check need for boundary zones;
M/Vl = 13.2/2.2(6) =1.0 < 3.0 (No boundary zones required)

Shear wall line 2 (Between grid lines B & H)
Shear wall line 2 resists shear forces from the upper roof, lower sloped areas and entrance diaphragms.
V_{upper} = 7.9k (35.1 KN)
$V_{low\ sloped\ roof}$ = 21.91k (97.5 KN)
$V_{entrance}$ = .81k (3.6 KN)
$V_{torsion}$ = 1 k (4.448 KN) (= 30% of longitudinal torsion force for 100% trans + 30% longit. load combination).

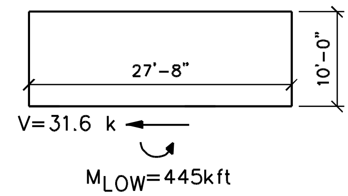


1 kip = 4.448 KN
1 kipft = 1.356 KNm
1 foot = 0.305m

1 kip = 4.448 KN
1 kipft = 1.356 KNm
1 foot = 0.305m



UPPER PORTION



LOWER PORTION

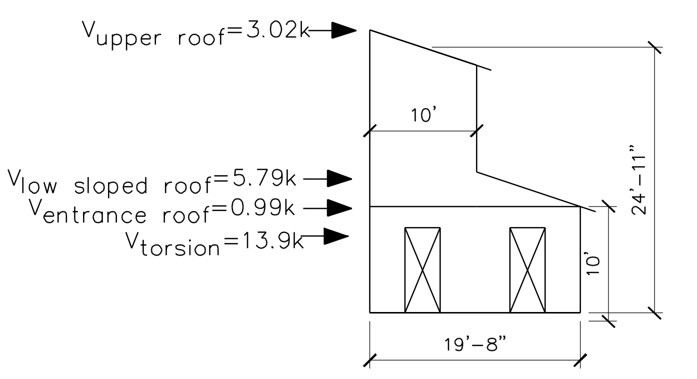
Shear strength of upper portion of wall (18')(12"/')(8")(174psi) = 301k (1348KN)> 7.9k (35.1 KN), OK
Shear strength of lower portion of wall (27.7')(12"/')(8")(174psi) = 463k (2059KN) >31.6k (141KN), OK

M_{upper} = (7.9k)(16.25') = 128 kft (174 KNm)
M/Vl = 128 / (7.9)(18) = 0.9 < 3.0 (no boundary zones required in upper wall portion)
M_{base} = (7.9)(26.25) + (21.91+0.81+1)(10') = 445 kft (603 KNm)
M/Vl = 445 / (24)(27.7) = 0.67 <3.0 (no boundary zones required in lower portion of wall)

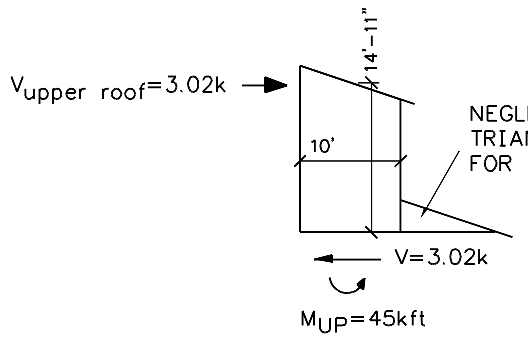
Shear wall line 7 (Walls 7A-7C & 7G-7I same by symmetry)

Shear wall line 7 resists shear forces from the upper roof, lower sloped areas and the sacristies.
V_{upper} = 3.015k (13.41 KN)
$V_{low\ sloped\ roof}$ = 5.79k (25.75 KN)
$V_{sacristy}$ = .99k (4.40 KN)
Torsion:
- From lower sloped roofs in transverse direction = 11.8 k
- From sacristy diaphragm in transverse direction = 0.68 k
- From lower sloped roof in longitudinal direction x 30% 3.2(0.30) = 0.96 k
- From sacristy in longitudinal direction x 30% = 1.53(0.30) = 0.46k
- Total torsion (for orthogonal effects = 13.9k (61.8 KN)

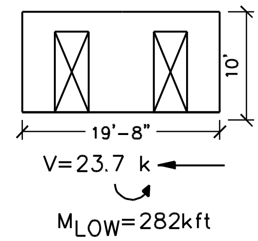
Total shear = 3.02 k + 5.79k + 0.99k + 13.9k = 23.7 k (105 KN)



SHEAR WALLS 7A–7C AND 7G–7I



UPPER PORTION



LOWER PORTION

Shear strength of upper portion of wall (10')(12"/')(8")(174psi) = 167k (743KN)> 3.02k (13.4KN), OK
M_{upper} = (3.02k)(14.92') = 45kft (61.0 KNm)
M/Vl = 45 / (3.02)(10) = 1.5 < 3.0 (no boundary zones required in upper wall portion)

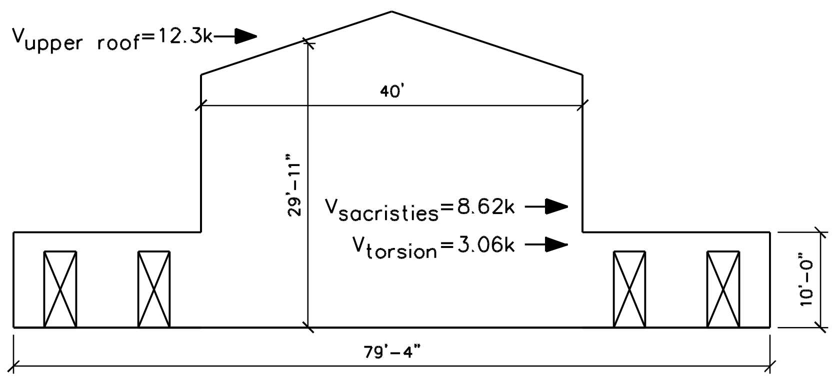
Distribute shear to wall piers in lower portion of wall based on relative rigidities;
R_{outer} = 960 kip / in R_{middle} = 4147 kip / in (previously calculated)
V_{outer} = 23.7k (960)/(2 x 960 + 4147) = 3.75 kips (16.7 KN)
V_{middle} = 23.7k (4147)/(2 x 960 + 4147) = 16.2 kips (72.1 KN)
Shear strength of outer pier = (3.25')(12"/')(8")(174) = 54.2 k (241 KN) > 3.75 k (16.68 KN), OK
Shear strength of middle pier = (6.5')(12"/')(8")(174) = 109 k (485 KN) > 16.2 k (72.1 KN), OK
M_{outer} = (8')(3.75k)/2 = 15 kft (20.34 KNm)
M/Vl = 15 / (3.75)(3.25') = 1.23 < 3.0 (no boundary zones required in outer piers)
M_{middle} = (8')(16.2k)/2 = 65 kft (88.14 KNm)
M/Vl = 65 / (16.2)(6.5) = 0.62 < 3.0 (no boundary zones required in middle pier)

Check overall action of lower portion of wall;

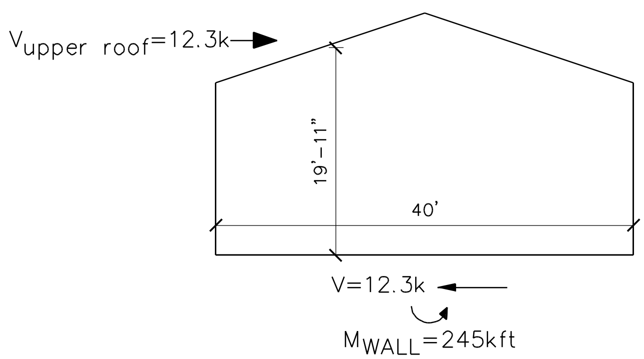
M_{base} = (3.02)(24.92) + (20.7)(10') = 282 kft (382 KNm)
M/Vl = 282 / (23.7)(19.67) = 0.61 <3.0 (no boundary zones required in lower portion of wall)

Shear wall line 8

Shear wall line 8 resists shear forces from the upper roof and the sacristies.
V_{upper} = 12.3k (54.7 KN)
$V_{sacristy}$ = 8.62k (38.3 KN)
Torsion: From sacristies in longitudinal direction x 30% =2 x 1.53(0.30) = 0.92k (4.09 KN)
Total shear = 12.3 k + 8.6k + 0.92 = 21.82 k (97.1 KN)

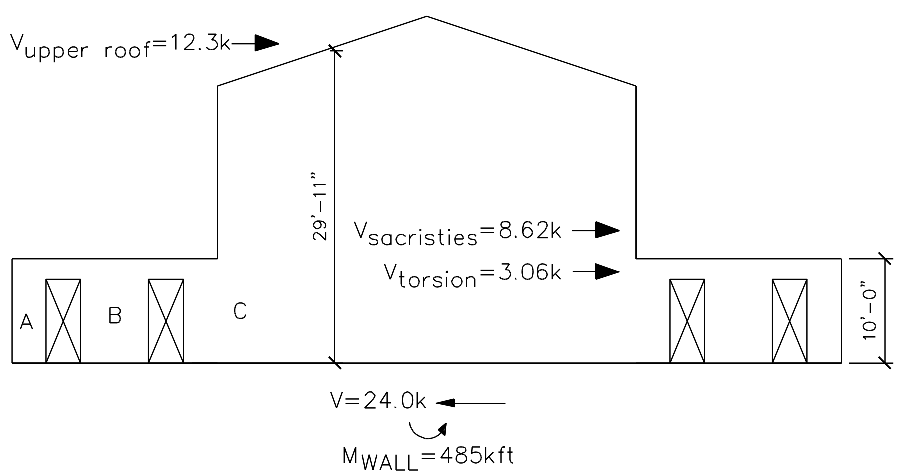


SHEAR WALL LINE 8



1 kip 4.448 KN
1 kipft = 1.356 KNm
1 foot = 0.305m

UPPER PORTION OF WALL



LOWER PORTION OF WALL

Shear strength of upper portion of wall (40')(12"/')(8")(174psi) = 668k (2971KN) > 12.3k (54.7KN), OK
M_{upper} = (12.3k)(19.92') = 245kft (332 KNm)
M/Vl = 245 / (12.3)(40) = 0.5 < 3.0 (no boundary zones required in upper wall portion)

Distribute shear to wall piers in lower portion of wall based on relative rigidities;
R_A = 960 kip / in R_B = 1018 kip / in R_C = 47802 kip / in (previously calculated)
ΣR = 2(960) + 2(1018) + 47802 = 51758 kips / in

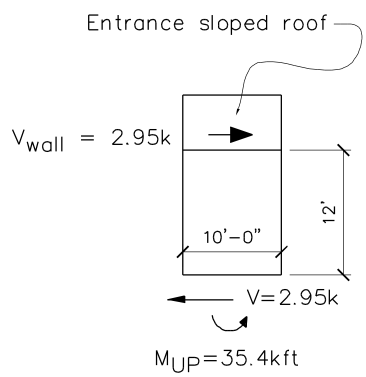
V_A = 24.0k (960)/(51758) = 0.45 kips (2.00 KN)
V_B = 24.0k (1018)/(51758) = 0.47 kips (2.09 KN)
V_c = 24.0k (47802)/(51758) = 22.2 kips (98.7 KN)
Shear strength of pier A= (3.25')(12"/')(8")(174) = 54.2k (241KN) > 0.45k (2.00 KN), OK
Shear strength of pier B= (3.33')(12"/')(8")(174) = 55.6k (247KN) > 0.47k (2.09 KN), OK
Shear strength of pier C= (46.5')(12"/')(8")(174) = 777 k > 22.2 k, OK
M_A = (8')(.45)/2 = 1.8 kft (2.44 KNm)
M/Vl = (1.8) / (0.45)(3.25) = 1.23 < 3.0 (no boundary zone required)
M_B = (8')(.47)/2 = 1.9 kft (2.58 KNm)
M/Vl = (1.9) / (0.47)(3.33) = 1.23 < 3.0 (no boundary zone required)
M_C = (8')(22.2)/2 = 89 kft (121 KNm)
M/Vl = 89 / (22.2)(46.5') = 0.09 < 3.0 (no boundary zones required in outer piers)

Check overall action of lower portion of wall;

M_{base} = (12.3)(29.92) + (11.68)(10') = 478 kft (648 KNm)
M/Vl = 478 / (24)(79.3) = 0.3 <3.0 (no boundary zones required in lower portion of wall)

Shear walls D1-D2 and F1-F2

Shear wall lines D & F resist force tributary to the entrance only.

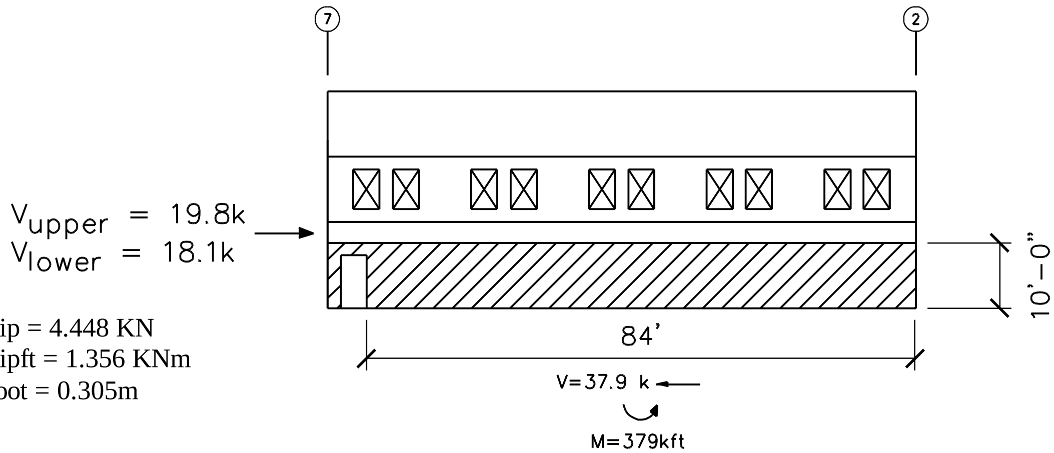


Shear Wall Line D1-D2 (Wall Line F1-F2 similar)

1 kip = 4.448 KN
1 kipft = 1.356 KNm
1 foot = 0.305 m

V_{wall} = 2.95 kips (13.12 KN)
M = Vl = (2.95)(12') = 35.4kft (48.0 KNm)
Shear strength of wall = (120")(8")(174) = 167 k (743 KN) > 2.95 k (13.1 KN), OK
Check need for boundary zones;
M/Vl = 35.4/2.95(10) =1.2 < 3.0 (No boundary zones required)

Shear walls A2-A7 and I2-I7



1 kip = 4.448 KN
1 kipft = 1.356 KNm
1 foot = 0.305m

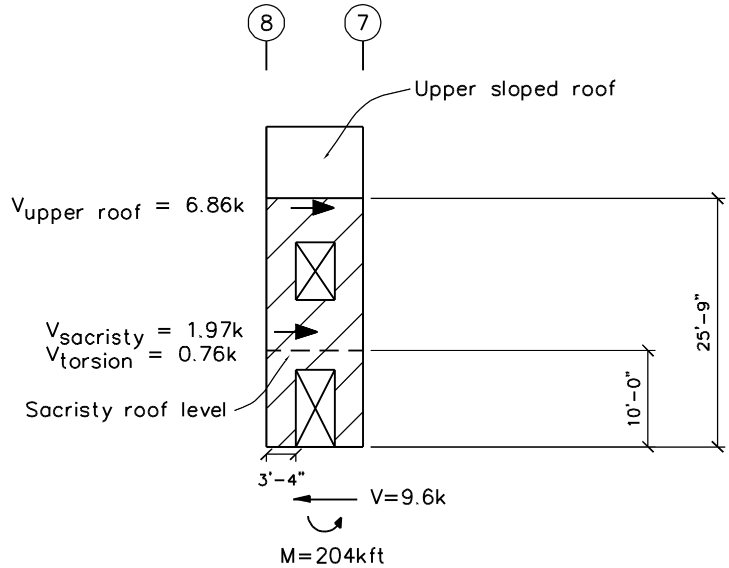
WALL LINE A2–A7 (WALL I2–I7 SIMILAR)

V_{wall} = 37.9 kips (169 KN)
M = Vl = (37.9)(10') = 379kft (514 KNm)
Shear strength of wall = (84 x 12")(8")(174) = 1403 k (6241 KN) > 37.9 k (169 KN), OK
Check need for boundary zones;
M/Vl = 379/37.9(84) =.12 < 3.0 (No boundary zones required)

Shear wall lines (C7-C8 and G7-G8)

Shear wall lines C & G resist shear forces from the upper roof and the sacristies.
V_{upper} = 6.86k (30.5 KN)
$V_{sacristy}$ = 1.97k (8.76 KN)
Torsion:
From sacristy in longitudinal direction = 0.673 k (2.99 KN)
From sacristy in transverse direction x 30% = 0.3(0.3kip) = 0.09kip (400N)
Total torsion (for orthogonal effects = 0.673 + 0.09 = 0.763k (3.39 KN)
Total shear = 6.86 k + 1.97k + 0.76k = 9.6 k (42.7 KN)

1 kip = 4.448 KN
1 kipft =- 1.356 KNm
1 foot = 0.305 m



Wall Line C7–C8 (Wall Line G7–G8 similar)

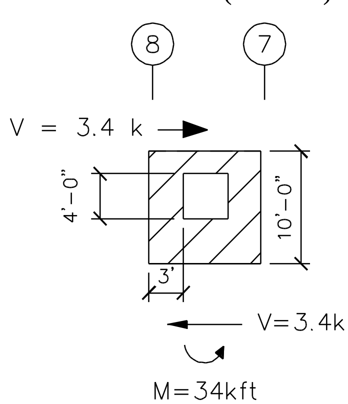
Distribute shear to wall piers in lower portion of wall (1/2 each)
V = ½(9.6) = 4.8 kips (21.3 KN)
Shear strength of pier = (3.33')(12"/')(8")(174) = 55.6 k (247 KN)> 4.8 k (21.3 KN), OK
M = (8')(4.8k)/2 = 19.2 kft (26.0 KNm)
M/Vl = 19.2 / (4.8)(3.33') = 1.2 < 3.0 (no boundary zones required in outer piers)

Check overall action of wall;

M_{base} = (6.86)(25.67) + (2.73)(10') = 204 kft (277 KNm)
M/Vl = 204 / (9.6)(10) = 2.12 <3.0 (no boundary zones required in lower portion of wall)

Shear wall lines A7-A8 & I7-I8

These walls resist shear forces from the sacristies.
V = 3.33k (14.8 KN)
Torsion:
From sacristy in transverse direction x 30% = 0.3(0.30) = 0.09k
Total shear = 3.33 k + 0.09k = 3.4 k (15.1 KN)



Wall Line A7–A8 (Wall Line I7–I8 similar)

Distribute shear to wall piers in lower portion of wall (1/2 each);
V = 3.4k/2 = 1.7 k (7.56 KN)
Shear strength of pier = (3')(12"/')(8")(174) = 49 k (218 KN) > 1.7 k (7.56 KN), OK
M = (4')(1.7)/2 = 3.4 kft (4.61 KNm)
M/Vl = 3.4 / (1.7)(3') = 0.67 < 3.0 (no boundary zones required in outer piers)

Check overall action of lower portion of wall;

M_{base} = (3.4)(10') = 34 kft (46.1 KNm)
M/Vl = 34 / (3.4)(10) = 1.0 <3.0 (no boundary zones required in lower portion of wall)

Typical Reinforcing; (See Figure 7-6 for typical reinforcing in concrete shear walls)

Use #5 bars @ 12" on center in horizontal and vertical direction for all concrete shear walls.

Development length per ACI 318 Sec. 12.2 $\frac{l_d}{d_b} = \frac{f_y \alpha\beta\lambda}{25\sqrt{f_c'}}$

$\alpha = 1.3$, $\beta = 1.0$ $\alpha\beta$ need not be greater than 0.8, use 0.8
$\lambda = 1.0$

$$\frac{l_d}{d_b} = \frac{60000(0.8)(1.0)}{25\sqrt{3000}} = 35$$

For # 5 bar l_d = 35*(5/8) = 22", use 24" (61 cm)

Use splice length = 1.3 l_d = 1.3(24") = 31.2", use 32" (81.3cm) for all splices.

Use 2 #5 vertical bars at ends of all wall segments and at openings.
Use 2 #5 horizontal bars typical above and below openings (extend bars 24" (61 cm) past edge of openings to develop bars) and continuously at top of walls for chord reinforcement.

(d) Horizontal bracing:
The bracing resist seismic forces only; load factor = 1.0
Design bracing for highest shear force (at shear wall line 2)
Shear to wall line 2A-2D passing through horizontal bracing = direct + torsion
V = 21.91 kips + 30%(3.2 kips) = 22.9 kips (102 KNm)
Axial (braces are at 45 degress); Axial force = 22.9 kips (1.41) = 32.3 kips (144 KN)

Try 4" Extra Strong Round Tubing (f_y = 36 ksi, r = 1.48 in., A = 4.41 in.^{2})

The perpendicular length of the brace is approximately 8', length of brace = 8'(1.41)=11.3'

$$\frac{KL}{\pi r}\sqrt{\frac{F_y}{E}} = \frac{(1.0)(11.3')(12''/')}{\pi(1.48)}\sqrt{\frac{36}{29000}} = 1.03$$

$$\phi_c F_{cr} = (0.85)(36\text{ksi})(0.658)^{\lambda^2} = 19.63\text{ksi } (135 \text{ N/mm}^2)$$

$\phi_c P_n$ = (19.63 ksi)(4.41 in.2) = 86.6 k (385 KN)

Check AISC Seismic Provisions; Design bracing as ordinary concerntrically braced frame (OCBF)

Slenderness: Bracing members shall have Kl/r $\leq 720/\sqrt{F_y} = 720/6 = 120$ (Section 14.2.a)
Kl/r = (1.0)(11.3)(12)/(1.48) = 92 < 120, OK

Required Compressive Strength of brace ≤ 0.8 times $\phi_c P_n$ (Section 14.2.b)
0.8$f_c P_n$ = (0.8)(86.6 k) = 69.3 k (308 KN) > 32.3 k (144 KN), OK

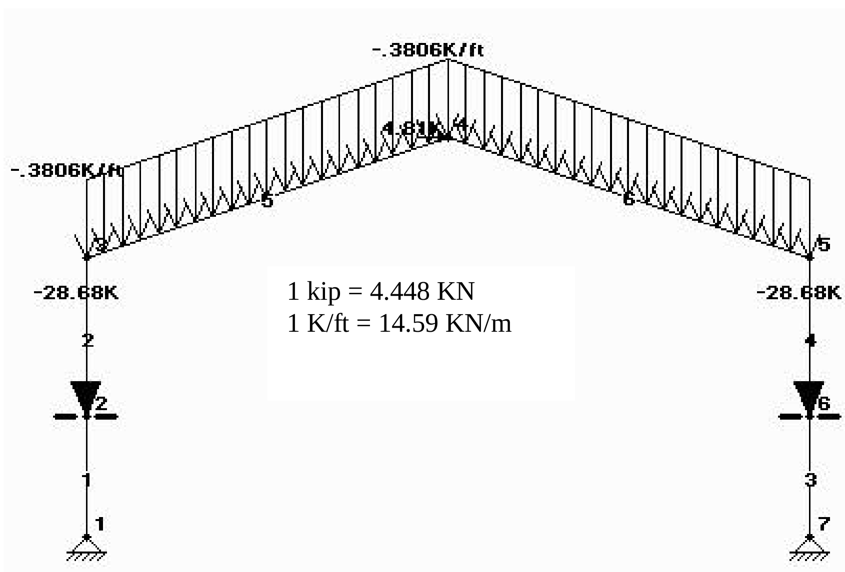
Width-to-Thickness Ratio: (Section 14.2.d)
$D/t \leq 1300/F^y = 1300/36 = 36.11$ (AISC Seismic Provisions Table I-9-1)
$D/t = 4.5 / 0.337 = 13.4 < 36.11$, OK

(e) *Moment frames*:
The moment frames resist seismic forces from the upper roof diaphragm and are braced by the lower sloped diaphragms (by horizontal bracing). The frames also support gravity loads from the upper roof and from the beam reactions along grid lines B & H.

Gravity:	Dead = 218 plf (3.18 KN/m)
	Live = 238 plf (3.47 KN/m)
	Beam reactions = 23.9 k (106.3 KN)
Seismic:	$V_{upper\ roof}$ =4.81 k (21.4 KN)

The members of the moment frames have already been checked for the gravity load combination and now are checked for the seismic load combination: $1.2D + 0.5L + 1.0E$ where $E = \rho Q_E + 0$. The vertical seismic effects are captured by the term $0.2\ S_{DS}D = 0.2(1.0) = 0.2D$. This term is added to the 1.2D load term to a total of 1.4D for this load combination.

Loads for elastic analysis: It is assumed that the lateral load to the upper portion of the frames is applied at the top middle node.



Beam Design:
The elastic analysis results show that the maximum moment (84.0 kft) and axial force in the beam (15.1 k) are lower than those for the load combination of 1.2D + 1.6L. By inspection it is seen that the beams are adequate.

Column Design:
Top portion: $\phi_c P_n$ = 802 k (3567 KN) $\phi_b M_n$ = 518 kipft (702 KNm)
Bottom portion: $\phi_c P_n$ = 927 k (4123 KN) $\phi_b M_n$ = 518 kipft (702 KNm)
For the upper portion of the column (above the horizontal bracing level) the maximum axial force = 13.2k (58.7 KN) and the moment is 84.0 kipft (114 KNm). For the lower portion of the columns the maximum axial force is 41.84 kips (186 KN) and the moment is 70.6 kipft (95.7 KNm). The columns are seen to be adequate by inspection.

Check AISC Seismic Provision Section 8.2 for columns;
Section 8.2 requires columns with high axial loads to be checked for increased demand.
$P_u / \phi P_n = 41.84 / 927 = 0.05 < 0.4$, OK

Check the colmn-beam moment ratio (AISC Seismic Provisions Section 9.6):

The relationship for the beam-to-column connection ratio, $\dfrac{\sum M^*_{pc}}{\sum M^*_{pb}} > 1.0$, is now checked:

$$\sum M^*_{pc} = Z_c(F_{yc} - P_{uc} / A_g) = 192(36 - 41.8 / 32.0) = 6661\text{kipin (752 KNm)}$$

$$\sum M^*_{pb} = 1.1(R_y M_p + M_v)$$

$M_p = 303$ kft (411 KNm)

The shears at the end of the beam when plastic hinges occur at the beam ends, V_p; (Note: The plastic hinge location must be verified to make sure they will occur at the toe of the haunch connections. See Problem H-4 for detailed calculation of plastic hinge locations. For this problem assume the platic hinges are located at the toe of the haunch.)
$V_p = 2M_p / L = 2(303) / 58$ ft = 10.4 kip (46.5 KN) assuming the hinges occur at the ends of the haunches.
The distance from the plastic hinge location to the connection centerline is approximatly 18" (45.7 cm).
$M_v = V_p(18") = 10.4k(18") = 187$ kipin. (16.3 KNm)
1.1(1.5(303)(12) + 187) = 6205kipin (701 KNm)
$M_{pc} / M_{pb} = 6661 / 6205 = 1.07$, OK

Check the limits of compactness for the column set forth in AISC Seismic Provisions Sec. 9.4b.
Limiting value from AISC Seismic Provisions Table I-9-1 = $52 / \sqrt{F_y} = 52 / 6 = 8.67$
For W14x109 b/t = 8.5 < 8.67, OK.

$$\text{Limiting value } \frac{520}{\sqrt{F_y}}\left[1 - 1.54\frac{P_u}{\phi_b P_y}\right] = \frac{520}{6}\left[1 - 1.54\frac{41.8}{(32\text{in}^2)(36\text{ksi})(0.9)}\right] = 81.3$$

For W14x109 $h/t_w = 21.7 < 81.3$, OK.

Check of the panel-zone of beam-to column connection:
Shear strength: (per AISC Seismic Provisions Sec. 9.3)
The required shear strength Ru of the panel-zone shall be determined by applying AISC Seismic Provsions Load Combinations 4-1 and 4-2 to the connected beam.

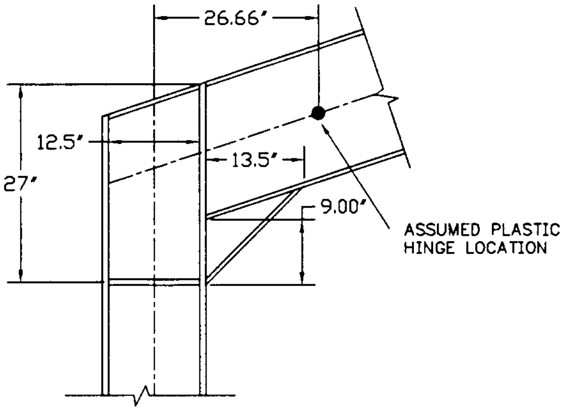
Governing Load Combination: $1.2D + 0.5L + \Omega_0 Q_E$ where $\Omega_0 = 3.0$ (AISC Seismic Provisions Eq. 4-1).
Note: The overstrength factor used for this check (3) is taken from the AISC Seismic Provisions. It is higher than the value required by FEMA 302 (2.5). The higher value is used to be conservative.

The maximum moment on the end of beam at the column interface due to seismic actions is 105 kipft (142 KNm). The shear to the joint is equal to this moment divided by the depth of the beam (including haunch) Ru = 105 kft / 27" = 47 kips (209 KN). The shear strength is determined from AISC Seimsic Provisions Equation 9-1:

$$\phi_v R_v = 0.75(0.6)F_y d_c t_p\left[1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right] = 0.75(0.6)(36)(14.32)(0.525)\left[1 + \frac{3(14.6)(0.86)^2}{18(14.32)(0.525)}\right] = 151\text{k (672 KN)}$$

151 kips (672 KN) > 47 kips (249 KN), Shear strength is OK.

Panel-Zone thickness:



1 inch = 25.4 mm

$t \geq (d_z + w_z) / 90$; (27" + 12.5) / 90 = 0.44 < 0.525, OK (AISC Seismic Eq. 9-2)

(f) Check beams along grid lines B & H that support concrete shear walls.

These beams act as collectors and chords for the low sloped roof areas in addition to supporting the concrete walls. They must be checked for the 1.2D + 0.5L + 1.0E load combination where $E = \Omega_0 Q_E + 0.2S_{DS}D$. The $0.2S_{DS}$ term adds 0.2 to the dead load factor. The collectors must be designed for $\Omega_0 = 2.5$. The beams act as collectors for longitudinal forces and chords for transverse forces.

Chord force = 32.3 kips (144 KN)   Collector force = 4.46 kips (19.8 KN)
100% longit. + 30% transv. = 2.5(4.46) + (0.30)(32.3) = 20.84 kips (92.7 KN)
30% longit. + 100% transv. = (0.30)(2.5)(4.46) + 32.3 = 36 kips (160 KN) (governs)

$w_u = 1.2D + 0.2D = 1.4D = 1.4(1325 \text{ plf}) = 1855 \text{ plf}$ (27.1 KN/m)
$M_u = w_u L^2/8 = (1855)(18')^2/8 = 75$ kft (102 KNm)

The compression flanges of the beams are completely supported by the concrete shear wall. It is assumed that the wall is connected to the beam by automatically welded studs @ 4' spacing (1.22m). Assume that the unsupported length for determing axial capacity is equal to KL = 4' (1.22m).

Determine axial capacity; W 14 x 38 (r_y = 1.55", A = 11.2 in^2, ϕM_p = 166 kft)
Kl/r = (48")/1.55" = 30.97
From AISC Table 3-36 ϕF_{cr} = 29.1 ksi (201 N/mm^2)
ϕP_n = (29.1 ksi)(11.2 in^2) = 326 kips (1450 KN)
$P_u / \phi P_n$ = 36 / 326 = 0.11 < 0.2, use interaction equation H1-1b

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}}\right) \leq 1 = \frac{36}{2(326)} + \left(\frac{102}{166}\right) = 0.67 < 1, \text{ OK}$$

By inspection assume that beam is OK in shear.

B-12 *Check allowable drift and PΔ effect*

The drift of the steel moment frames is checked due to their low rigidity as compared to that of the concrete shear walls. It is assumed that the concrete shear wall drift is within the limits specified in Table 6-1. The interstory drift is found by adding the displacement of the moment frame at the top center node and adding that to the deflection of the shear walls at the low sloped roof areas.
Deflection of wall 2A-2D: Shear = 31.6 k (141 KN) = deflection = V/R R = 31.6 / 18920 k / in. ≈ 0.
Deflection of moment frames at center of gravity of story (from elastic analysis) = 0.11 inches (2.8 mm)
The story deflection is determined by multiplying this deflection by C_d.

Story deflection = $C_d(0.11") = 5(0.11) = 0.55$ inches (14mm)
Allowable Deflection = 0.02(33.3') = 8" (203 mm) > 0.55" (14mm), OK (From Table 6-1)

D-1 *Check for Performance Objective 2A (Safe Egress).*
This performance objective uses the same ground motion as the Life Safety Performance Objective (1A). The structure is a one-story building analyzed by the ELF procedure, and therefore, the seismic effects, Q_E, in step B10 may be scaled up in a linear manner.

D-2 *Determine the pseudo lateral load,* $V=C_1C_2C_3S_aW$
C_1: Modification factor to relate expected maximum inelastic displacement to displacements calculated for linear elastic response. (per FEMA 273 Section 3.3.1.3)
$T_0 = S_{D1}/S_{DS} = 0.75$
T = Fundamental period of building = 0.25 seconds
$C_1 = 1.5 – (0.25\text{-}0.10)/(0.75\text{-}0.10)*(0.5) = 1.38$

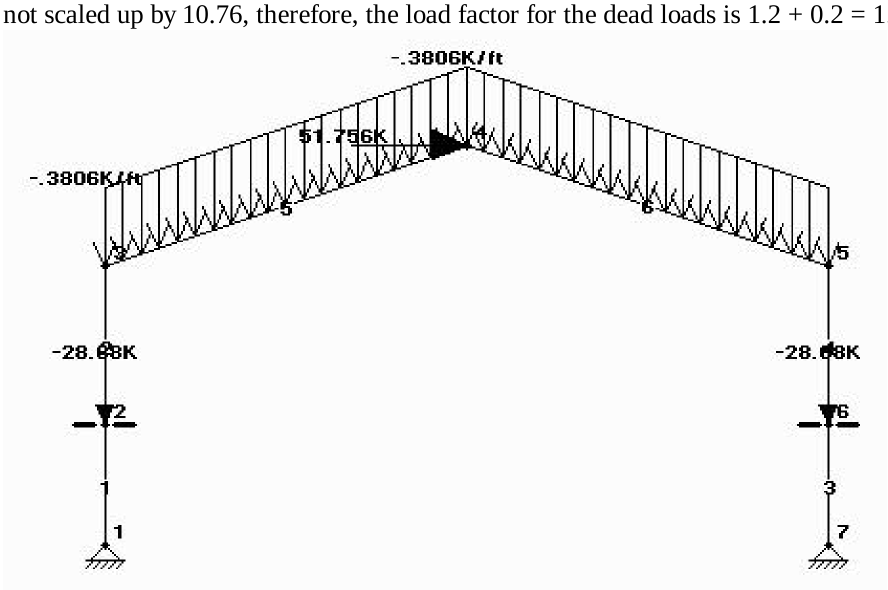
C_2: Hysterisis modification factor, from Table 5-2;
$C_2 = 1.3$ (Framing Type 1, Life Safety and T = 0.1 sec)

C_3: Modification factor to account for P-delta effects.
Assume that the building exhibits positive post-yield stiffness.
$C_3 = 1.0$ for positive post-yield stiffness.

V = (1.38)(1.3)(1.0)(1.0g)(635 k) = 1139 k (5066 KN)

D-3 *Determine seismic effects.*
The seismic effects in Steps B-4 through B-9 are scaled up by the factor R x C1 x C2 x C3.
Scale factor = (6)(1.38)(1.3)(1.0) = 10.76

D-4 *Determine the combined load effects.*
The shear force to the wall segments are scaled up by the factor 10.76 and the resulting shear demand is checked against the shear strength multiplied by the appropriate m-factor from Table 7-4.
The moment frames will be analyzed for the load combination 1.2D + 0.5L + E, where the E term represents the seismic actions determined from step B.10 scaled up by 10.76 (Note: the term $0.2S_{DS}$ x D is not scaled up by 10.76, therefore, the load factor for the dead loads is 1.2 + 0.2 = 1.4)



Horizontal Bracing:

The axial force in the horizontal pipe bracing is scaled up by 10.76 to 10.76(32.3) = 344 k (1530 KN). The bracing enhanced performance objectives will be checked as if the bracing were a concentrically braced frame.

Collectors / Chords along grid lines B & H
These beams act as collectors and chords for the low sloped roof areas in addition to supporting the concrete walls. Scale up forces by 10.96
Chord force = 32.3 kips (10.96) = 354 k (1575 KN)   Collector force = 4.46 k (10.96) = 48.7 k (217 KN)

$w_u = 1.2D + 0.2D = 1.4D = 1.4(1325 \text{ plf}) = 1855 \text{ plf } (27.1 \text{ KN/m})$
$M_u = w_uL^2/8 = (1855)(18')^2/8 = 75 \text{ kft } (102 \text{ KNm})$

D-5 *Identify force-controlled and deformation controlled structural components.*
The concrete shear walls have very low axial load demands. Footnote 1 of Table 7-3 requires that the axial load be less than 0.15 $A_g f'_c$ to be governed by shear. The maximum shear stress in all of the wall is less than $6\sqrt{f_c'}$ and therefore, the walls are checked as deformation controlled structural components.
The steel moment frames are checked as deformation controlled components.
The horizontal bracing is checked as a deformation controlled component.

The connection of the horizontal bracing to the shear walls along lines 2 & 7 and the collectors along lines B & H are checked as force controlled actions.

D-6 *Determine Q_{UD} and Q_{CE} for deformation controlled components*

Shear Wall Segments

The highest demand / capacity ratio to any wall pier element from step B.11 is 16.2k / 109k = 0.15 (for the lower portions of wall line 7). Only this element is checked;

Q_{UD} = (10.76)(16.2 k) = 174.3 kips (775 KN)
Q_{CE} = 109 kips (485 KN) (determined previously)

Moment Frames

The expected strength of the steel members is based on $F_{ye} = R_yF_y = 1.5(36) = 54$ ksi (372 N/mm^2) and Z for the section.

Beams;
Moment Strength = ZF_{ye} = 101(54) = 455 kft (617 KNm)
Shear Strength = $0.6F_{ye}(d \times t_w)$ = 0.6(54)(17.99" x 0.355") = 207 k (921 KN)
Axial Strength = 340 kips (this is for 36 ksi; scale up by 54/36 to obtain F_{YE} strength)
Axial Strength = (340 k)(54/36) = 510 k (2268 KN)

Moment Demand = 180 kft (244 KNm)
Shear Demand = 11.8 k (52.5 KN)   (Elastic analysis not shown)
Axial Demand = 41 k (182 KN)

Columns;

Moment Strength = ZF_{ye} = 192(54) = 864 kft (1179 KNm)
Shear Strength = $0.6F_{ye}(d \times t_w)$ = 0.6(54)(14.32" x 0.525") = 244 k (1085 KN)
Axial Strength for 36 ksi column: Top segment = 802k; Bottom segment = 927k
Scale up to F_{YE} strength: Top segment = (802k)(54/36) = 1203 k (5351 KN);
Bottom segment = (927k)(54/36) = 1391k (6187 KN)

Moment Demand = 286 kft (388 KNm) (at horizontal bracing level)
Shear Demand = 35.14 k (156 KN) (Elastic analysis not shown)
Axial Demand @ top portion = 24.18 k (108 KN)
Axial Demand @ bottom portion = 52.86 k (235 KN)

Panel Zones;

The maximum moment on the end of beam at the column interface due to seismic actions is 179.8 kipft (244 KNm). The shear to the joint is equal to this moment divided by the depth of the beam (including haunch) = 179.8(12)/27 = 80 kips (356 KN). The shear strength is determined from AISC Seimsic Provisions Equation 9-1:

$$\phi_v R_v = 0.75(0.6)F_{ye}d_c t_p\left[1+\frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right] = 0.75(0.6)(54)(14.32)(0.525)\left[1+\frac{3(14.6)(0.86)^2}{18(14.32)(0.525)}\right] = 227\text{k (1010 KN)}$$

D-7 *Determine DCR's for deformation-controlled components and compare with allowable m-values for Safe Egress*

Shear Walls

The highest demand / capacity ratio to any wall pier element from step B.11 is 16.2k / 109k = 0.15 (for the lower portions of wall line 7). Only this element is checked;

Q_{UD} / Q_{CE} = 174.3 / 109 = 1.60

From Table 7-3 for concrete shear wall segments controlled by shear: m = 2.0.
1.60 < 2.0, OK

Steel Moment Frames

The m-factors for fully restrained moment resisting frames is taken from Table 7-12. The m-factor for all actions is 4.

Beams;

Moment DCR = 180 / 455 = 0.4 <4, OK
Shear DCR = 11.8 / 207 = 0.06 <4, OK
Axial DCR = 41 / 510 = 0.08 <4, OK

Columns;

Moment DCR = 1022 / 864 = 1.18 <4, OK
Shear DCR = 75.6 / 244 = 0.31 <4, OK
Top portion Axial DCR = 24.2 / 1203 = 0.02 <4, OK
Bottom portion Axial DCR = 52.9 / 1391 = 0.04 < 4, OK

Panel Zones;

The m factor for panel zones from Table 7-12 is 4.8 for Safe Egress
Demand / Capacity = 80 / 227 = 0.35 < 4.8 OK

Horizontal Bracing

The m-factor for the horizontal bracing from Table 7-10 is 2.9 for braces in compression and $\frac{d}{t} \le \frac{1500}{F_y}$.

Axial Demand = 344 kips (1530 KN)
Axial Capacity = 86.6 kips (this is based on 36 ksi; scale it up by 54/36 for enhanced performance objectives)
Axial Capacity = (86.6 k)(54/36) = 130 kips (578 KN)
Axial DCR = 344 / 130 = 2.65 < 2.9, OK

D-8 *Determine Q_{UF} and Q_{CL} for force-controlled components and compare Q_{UF} with Q_{CL}*

Note: Q_{CL} contains the appropriate strength reduction factor per paragraph 6-3a(3)b

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3 J} \qquad \text{(Eq. 6-4a)}$$

where $J = 1.0 + S_{DS} = 1.0 + 0.2 = 1.2$ and C_1, C_2 and C_3 have been determined previously.

Therefore, the scale factor for QE is;

$$SF = \frac{1}{C_1 C_2 C_3 J} = \frac{1}{(1.38)(1.3)(1.0)(1.2)} = 0.46$$

Chord / Collector elements along grid lines B & H

Worst case is for the chord force:
Q_E: Chord force =354 kips (1574KN)
Q_G = 1.4D = 1855 plf (27.06 KN/m), M_u = 75 kft (102 KNm) (determined previously)

$Q_{UF} = SF(Q_E) = (0.46)(354) = 163$ kips (725 KN) axial and M_u = 75 kft (102 KNm)

ϕP_n = 326 kips (1450 KN) ϕM_p = 166 kft (225 KNm) (determined previously)

$P_u / \phi P_n = 163 / 326 = 0.5 > 0.2$ Use AISC LRFD equation H1-1a

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_u}{\phi M_p}\right) = \frac{163}{326} + \frac{8}{9}\left(\frac{75}{225}\right) = 0.80 < 1.0\text{, OK}$$

Shear wall to horizontal bracing connections.

These connections are designed in Section D-9 based on the capacity of the brace. The design force is greater than the force that results from Q_{UF}. Therefore, assume connections are OK for force-controlled action.

D-9 *Revise member sizes, as necessary, and repeat analysis.*
No member sizes need to be revised. Design of steel connections is done now;

Moment Connections:

Determine if continuity plates are required:

Determine demand;
$P_{bf} = A_f F_{ye} = A_f R_y F_y = (0.57)(7.495)(1.5)(36) = 231$ kips

Determine capacity;

$$\phi R_n = \phi\left[(2.5k + N)F_{yw}t_w + A_{st}F_{yst}\right]$$

$$A_{st} = \frac{P_{bf} - (2.5k + N)F_{yw}t_w}{F_{yst}} = \frac{231 - (2.5(1.56") + (0.57"))(36ksi)(0.525")}{36ksi} = 4.07 \text{ in}^2\ (26.3 \text{ cm}^2)$$

Design stiffeners in accordance with AISC LRFD Sec. K.9;

$$b_{st} + \frac{t_{cw}}{2} \geq \frac{b_b}{3} \rightarrow b_{st} \geq \frac{b_b}{3} - \frac{t_{cw}}{2}$$

where b_{st} = width of single stiffener, b_b = width of beam flange, t_{cw} = thickness of column web

$$b_{st} = \frac{7.495}{2} - \frac{0.525}{2} = 3.48" \text{ (88mm), use 4.5" (114mm)}$$

$t_{st} \geq \frac{t_{bf}}{2}$ where t_{st} = thickness of a single stiffener, t_{bf}= thickness of beam flange

$$t_{st} = \frac{.57}{2} = 0.285" \text{ (7.24mm), use 1/5" (12.7mm)}$$

Use 4.5" x ½" (114 mm x 88mm) stiffeners on both sides of column. (Area = 2 x 4.5 x ½ = 4.5 in^2 > 4.07 in^2)

Design welds for the stiffeners;
Stiffener to column web;

$F = P_{bf} - (2.5k + N)F_{yw}t_w = 231 - (2.5(1.56) + 0.57)36(0.525) = 147$ k
Assume minimum weld size = 3/16" (4.8mm)
Strength of weld = 0.75(0.6)(70)(0.707)(3/16) = 4.18 kip / in (0.73 KN/mm)
Length of weld required = 147 / 4.18 = 35.2" (89.4 cm).
The length of weld available = 11.25" x 2 sides x 2 stiffeners = 45" (114 cm) > 35.2"
Check shear strength of the base material;

$\phi R_n = \phi R_{nw} = \phi(0.6F_u)t = (0.75)(0.6)(58)(0.525)2 = 27.4$ kips / in (4.8KN/mm) > 4.18 kips / in

Stiffener to column flange; Use full penetration groove welds

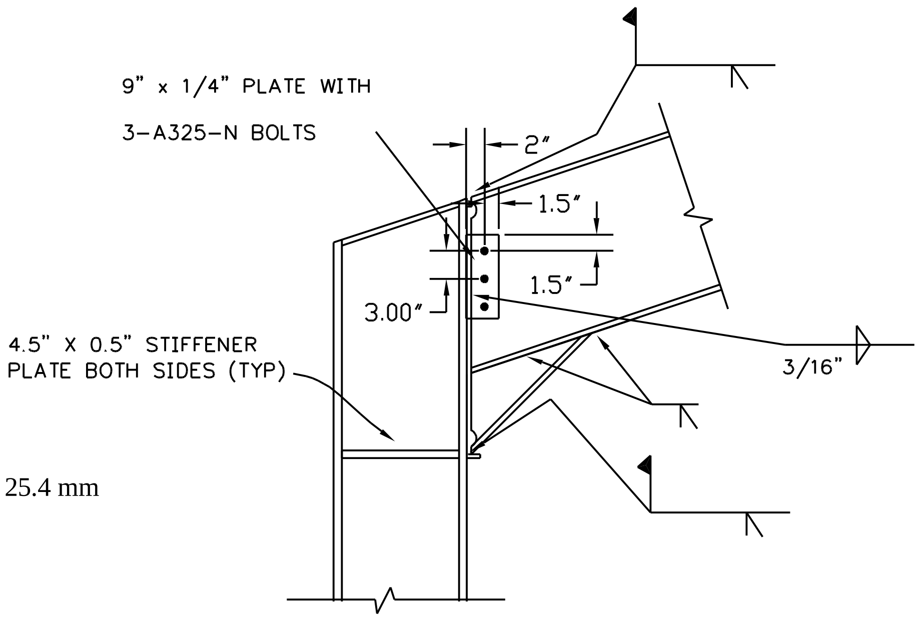
Design the single-plate web connection;

The governing load combination = $1.2D + 1.6L_r + 0.5L$
w_u = 1.2(218 plf) + 1.6(238 plf) = 642 plf (9.37 KN/m)
$V_u = w_uL/2$ = (642 plf)(60')/2 = 19.3 k (85.8 KN)

From AISC LRFD Table 9-10; assuming the column provides a rigid support, for ¾" diameter A325-N bolts and single-plate material with F_y = 36 ksi and F_u = 58 ksi, select three rows of bolts, ¼" single-plate thickness, and 3/16" fillet weld size: ϕRn = 27.8 kips (124KN)

Check supported beam web: From Table 9-2, for three rows of bolts, beam material with Fy = 36 ksi and Fu = 58 ksi, and L_{ev} = 1-1/2" and L_{eh} = 1-1/2" (Assumed to be 1-1/4" for calculation purposes to account for possible underrun in beam length),
ϕRn = 235 kips / inch (0.355) = 83.4 kips (371 KN) > 27.8 kips (124KN)

Use ¼" thick plate (6.35mm) f_y = 36 ksi (248 N/mm^2) with three ¾" diameter A325-N bolts. Weld plate to column with 3/16" (4.76 mm) welds along entire plate length on both sides.



1 inch = 25.4 mm

Design of horizontal bracing connections:

The design of these connections follows Figure 7-22.
Assume plate thickness = ½" (12.7 mm); thickness of brace = 0.337" (8.56mm); E70XX welds

Design of brace-to-gusset weld;

Design weld capacity to be greater than axial capacity of brace = $R_yF_yA_g$ = 1.5(36ksi)(4.41 in.^{2}) = 238 k (1059 KN)
Minimum weld size = 3/16" (4.76mm) (AISC LRFD Table J2.4)
Maximum weld size = brace thickness – 1/16" = 0.337"-1/16" = 0.28" (7.11mm) (AISC LRFD Sec. J2.b)
Use ¼" welds (0.25") (6.35mm) 3/16<1/4<0.28, OK

Strength of weld; (per AISC LRFD Sec. J.4 and Table J2.5)

Weld material: $\phi R_{nw} = \phi(t_e)$ (0.6 F_{EXX}) = 0.75(0.6)(70ksi)(0.707)(0.25) = 5.6 kips / inch (0.98 KN/mm) (governs)
Base material: $\phi R_{nw} = \phi(0.6F_u)t$ = (0.75)(0.337")(0.6)(58) = 8.8 kips / in (1.54 KN/mm)

Length of weld required = 238 k / (5.6 k/") = 43" (109cm) (4 welds at connection, use 11" (27.9 cm) welds, 4 x 11 = 44 > 43)

Use 11" (27.9 cm) long ¼" (6.35 mm) fillet welds on all four sides

Check gusset plate capacity

Tension rupture of plate: The tension rupture strength of the plate is based on Whitmore's area. This area is calculated as the product of the plate thickness times the length W, shown in the sketch as a 30 degree angle offset from the connection line. The tension rupture strength of the plate is designed to exceed the tensile strength of the brace, 238 kips.

W = 2(11"*tan 30) + 4.5" = 17.2" (43.7 cm)
$\phi_t P_n = \phi t F_u A_e$ = 0.75(58)(17.2")(0.5") = 374 k (1664 KN) > 238k (1058 KN) AISC LRFD Eq. D1-2

Block shear rupture strength of plate:
$\phi R_n = \phi(0.6F_yA_{gv} + F_uA_{nt})$ AISC LRFD Eq. J4-3a
ϕR_n = 0.75(0.50)[(0.6)(36)(2 x 11"/cos 30) + (58)(17.2)] = 528 k (2349 KN) > 238 k (1058 KN)

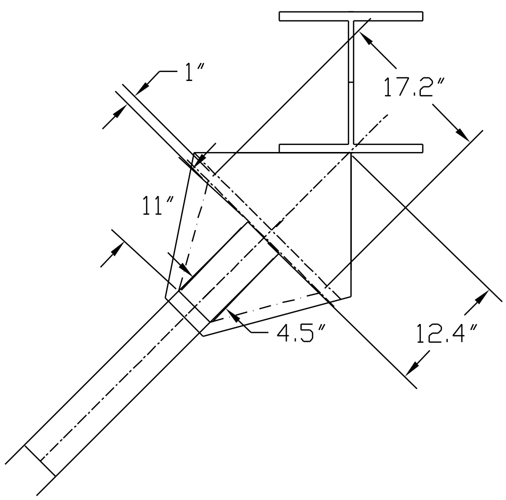
$\phi R_n = \phi[0.6F_uA_nv + F_yA_{gt}]$ AISC LRFD Eq. J4-3b
$\phi R_n = 0.75(0.5)\ [(0.6)(58)(2 \times 11) + (36)(17.2)] = 519$ k (2309 KN) > 238 k (1058 KN)

Buckling of plate:

Buckling capacity of the brace = $A_gF_{cr} = \phi_cP_n / \phi_c$ = 87 kips / 0.85 = 102 kips (454 KN) (buckling strength determined previously)

$$0.90\frac{4000t^3\sqrt{f_y}}{l_c} = 0.90\frac{4000(1/2)^3\sqrt{36}}{12.4} = 218 \text{ kips (970KN)} > 102\text{kips (457 KN)}\text{ , OK}$$

Out-of-plane strength of plate: The bracing member can buckle both in and out of plane due to the round section used. For out-of-plane buckling the gusset plate must be able to accommodate the rotation by bending. The brace shall terminate on the gusset a minimum of two times the gusset thickness from the theoretical line of bending which is unrestrained by the column or beam joints. This ensures that the mode of deformation in the plate will be through plastic hinging rather than torsional fracture.



1 inch = 25.4 mm

Design of gusset-to-column flange and beam web weld;

This connection requires a weld length greater than the column flange. The plate must be welded to both the column flange and the beam web to develop the brace force.
Design weld capacity to be greater than horizontal component of brace capacity = (0.707)(238) = 168 kips (747 KN)
Column flange thickness = 0.86 in. (21.8mm)
Beam web thickness = 0.31”
Minimum weld size = ¼” (6.35mm) (AISC LRFD Table J2.4)
Maximum weld size = beam web thickness ≈ 0.25” (6.35 mm)
Use 1/4” (6.35 mm) welds

Strength of weld; (per AISC LRFD Sec. J.4 and Table J2.5)

Weld material: $\phi R_{nw} = \phi(t_e)(0.6\ F_{EXX})$ = 0.75(0.6)(70ksi)(0.707)(0.25) = 5.57 kips / inch (0.98 KN/mm) (governs)

Base material: $\phi R_{nw} = \phi(0.6F_u)$ = (0.75)(0.31”)(0.6)(58) = 8.1 kips / in (1.42 KN/mm)

Length of weld required = 168 k / (5.57 k/”) = 30.16” (76.6 cm) (2 welds at connection weld 16” (40.64 cm) long = 32” > 30.16”)

Use 1/4” (6.35mm) fillet welds on top and bottom of plate.

Design member to develop force into shear wall

Vertical component = (0.707)(238k) = 168 kips (747 KN)

Use 6 x 6 x ½ angle to develop forces; Assume 7/8” anchor bolts to concrete shear wall

Check gross section yielding; $P_u = \phi_t F_y A_g$ = (0.9)(36)(5.75) = 186 kips (827 KN) > 168 kips (747 KN)
Check net section fracture; $P_u = \phi_t F_u A_e$ = (0.75)(58)(0.85)(5.75 – (7/8+1/16)(0.5)) = 195 kips (867 KN) > 168 kips

Design weld of plate to angle
Minimum weld size = ¼” (6.35mm) (AISC LRFD Table J2.4)
Maximum weld size = plate thickness – 1/16” = 0.5-1/16 = 0.44” (11.18mm)
Use 7/16” (11.11 mm) welds (0.438”)

Strength of weld; (per AISC LRFD Sec. J.4 and Table J2.5)

Weld material: $\phi R_{nw} = \phi(t_e)(0.6\ F_{EXX})$ = 0.75(0.6)(70ksi)(0.707)(0.438) = 9.75 kips / inch (1.71 KN/mm) (governs)

Base material: $\phi R_{nw} = \phi(0.6F_u)t$ = (0.75)(0.5”)(0.6)(58) = 13.05 kips / in (2.29 KN/mm)

Length of weld required = 168 k / (9.75 k/”) = 17.23” (43.8 cm) (2 welds at connection, weld for 16” on both sides of plate. Total length = 2 x 16 = 32” (81.3 cm) > 17.23” (43.8 cm), OK

Use 16” long (81.3 cm) 7/16” (11.11mm) fillet welds along top and bottom of plate.

Design bolts for angle to wall connection.

Design per FEMA 302

Tensile strength of bolts:

-Assume 7/8” diameter bolts @ 12”o/c with a 6” embedment length

-Assume that the edge distance for anchors > than 6" = embedment length

Strength based on steel: $P_s = 0.9A_bF_un$ (FEMA 302 Eq. 9.2.4.1-1)

$P_s = (0.9)(0.6\text{in.}^2)(58 \text{ ksi})(1) = 31.3$ kips (139 KN)

Strength based on concrete: $\phi P_c = \phi\lambda\sqrt{f_c'}(2.8A_s)n$ (FEMA 302 Eq. 9.2.4.1-2)

$\phi P_c = (0.65)(1.0)\sqrt{3000}(2.8)(\pi)(6)^2(1) = 11.3$ kips (50.3 KN) (governs)

Shear strength of bolts:

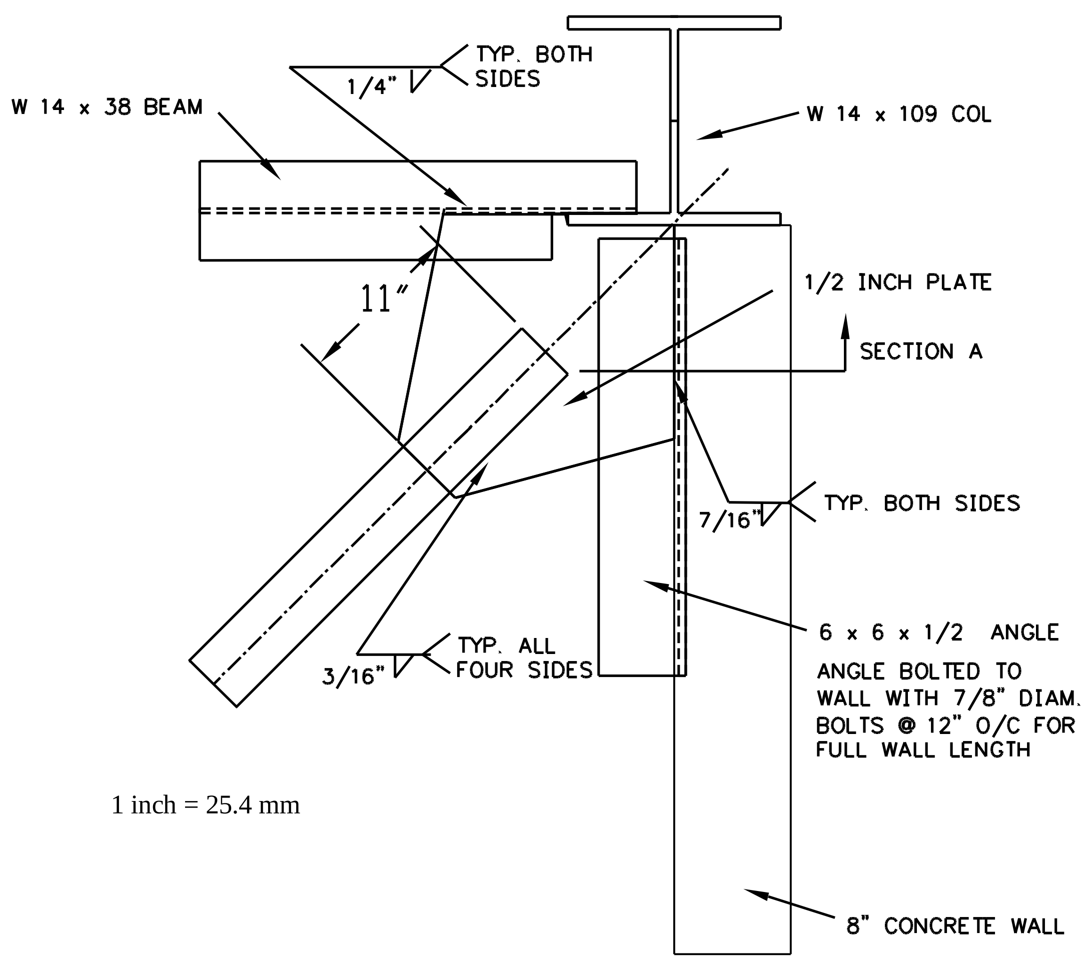
Strength based on steel: $V_s = 0.75A_bF_un$ (FEMA 302 Eq. 9.2.4.2-1)

$V_s = (0.75)(0.60)(58)(1) = 26.1$ kips (116 KN)

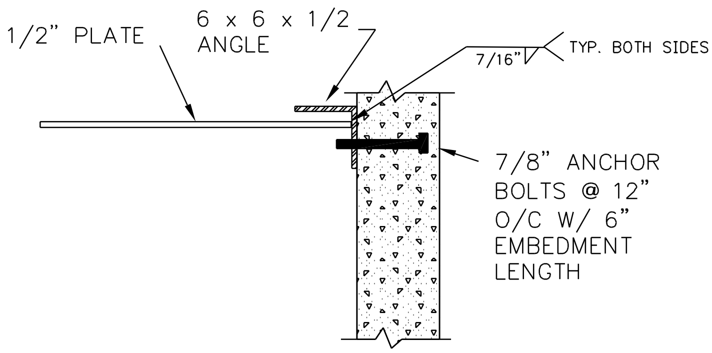
Strength based on concrete: $\phi V_c = \phi 800A_b\lambda\sqrt{f_c'}n$ (FEMA 302 Eq. 9.2.4.2-2)

$\phi V_c = (0.65)(800)(0.6)(1.0)\sqrt{3000}(1) = 17.1$ kips (76.1 KN) (governs)

Shear force demand = vertical component of brace capacity = 168 kips (753 KN)
Number of bolts required = 168 / 17.1 = 9.8 bolts, use 10 bolts @ 12" (0.31m) on center.



1 inch = 25.4 mm



SECTION A

Design of gusset-to-column weld;

Design weld capacity to be greater than brace capacity = 238 k (1059 KN)
Column flange thickness = 0.86 in. (21.8mm)
Minimum weld size = ¼" (6.35mm) (AISC LRFD Table J2.4)
Maximum weld size = plate thickness – 1/16" = 0.5-1/16 = 0.44" (11.18mm)
Use 7/16" (11.11 mm) welds (0.438")

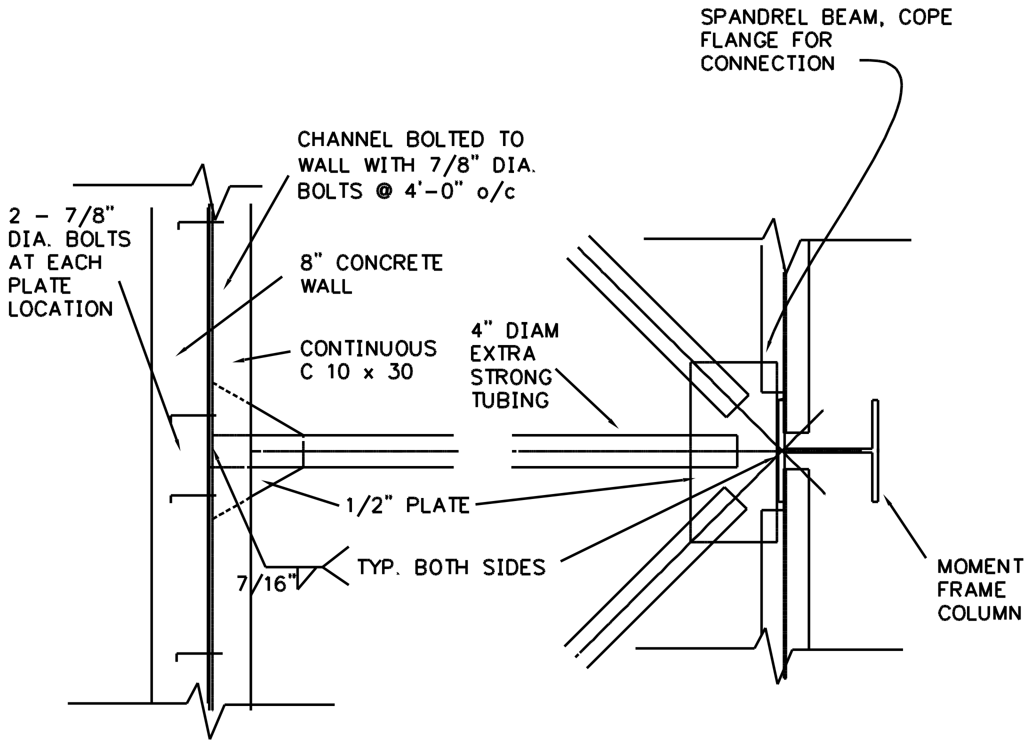
Strength of weld; (per AISC LRFD Sec. J.4 and Table J2.5)

Weld material: $\phi R_{nw} = \phi(t_e)(0.6\ F_{EXX}) = 0.75(0.6)(70ksi)(0.707)(0.438) = 9.75$ kips / inch (1.71 KN/mm) (governs)

Base material: $\phi R_{nw} = \phi(0.6F_u)t = (0.75)(0.5")(0.6)(58) = 13.05$ kips / in (2.29 KN/mm)

Length of weld required = 238 k / (9.75 k/") = 24.4" (62 cm) (2 welds at connection, weld for full length of column flange = 14.61" (37.11 cm) along top and bottom of plate. Total length = 2 x 14.61 = 29.22" (74.2 cm) > 24" (43.8 cm), OK

Use 7/16" (11.11mm) fillet welds along entire length of column flange on top and bottom.



PLAN OF HORIZONTAL BRACING AT LOW ROOF

1 inch = 25.4 mm